PDHonline Course E541 (4 PDH)

Transmission Line Design - Volume 5

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Transmission Line Design Volume V – Pole Structures

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This course is based on a USDA document, "Design Manual for High Voltage Transmission Lines", United States Department of Agriculture, Rural Utilities Service December, 2015.

Introduction

The primary purpose of this series of courses is to furnish engineering information for use in designing transmission lines. Good line design should result in high continuity of service, long life of physical equipment, low maintenance costs, and safe operation. These courses presents a generalized "how to" guide for the design of a high voltage transmission line.

The engineering information in this course is for use in design of transmission lines for voltages 230 kV and below. Designs should be adapted to various conditions and local requirements. Engineers should investigate local weather information, soil conditions, operation of existing lines, local regulations, and environmental requirements and evaluate known pertinent factors in arriving at design recommendations.

This course is based on the requirements of the National Electrical Safety Code® (NESC®). However, since the NESC is a safety code and not a design guide, additional information and design criteria are provided in this course as guidance to the engineer. The additional design criteria are based on practices of many utilities in the United States.

This series includes five volumes. For the best understanding of the material, they should be studied in order. The volumes are generally divided into the following categories.

Volume I. This volume is an introduction to transmission line design and addresses siting issues, plan and profile drawings, loading, and distribution underbuild.

Volume II. This volume is all about clearances. Ground clearances, horizontal clearances, clearances from other live parts, and clearances to supporting structures are addressed.

Volume III. This volume discusses the materials involved in transmission line design and construction including insulators, conductors and hardware.

Volume IV. This volume in the series is concerned with the structural aspects of transmission line design and includes foundations and guyed structures.

Volume V. The final volume in the series is concerned with the structural aspects of transmission line design and includes single-pole structures and H-frame structures.

Chapter 1

Single Pole Structures

During preliminary planning stages of lines above 161 kV, studies should be made to evaluate the economics of different types of structures as related to conductor size. In most instances, for lines of 230 kV and below, wood structures have historically been the economical choice.

However, in more heavily loaded situations (larger wires, longer spans) steel and prestressed concrete structures may be more economical than wood, especially considering the long-term maintenance costs associated with wood structures. In some instances, other types of material have been used because of environmental or meteorological constraints. For voltages 345 kV and above, it may be difficult to obtain long span construction utilizing wood, due to height or strength reasons.

In most instances, for lines 230 kV and below, an economic study can help to determine structure configuration, base pole class (wood, steel or prestressed concrete) and height.

Factors which limit structure spans include:

- Strength: Horizontal spans are limited by crossbrace, poles, etc. Vertical spans are limited by crossarms, post insulators, and structure strength. For H-frame structures, horizontal and vertical spans are also limited by pullout resistance for H-frame structures.
- Conductor Separation: Conductor separation is intended to provide adequate space for line crew personnel on poles, prevention of contact and flashover between conductors.
- Clearances-to-Ground: Limits on spans are directly related to height of structures.
- Insulator Swing: The ratio of horizontal to vertical span will be limited by insulator swing and clearance to structure.

Historically, preliminary cost estimates have been usually based on level ground spans. With the advent of computer-automated line design and optimization software, preliminary cost estimates can now be performed using a preliminary profile digitized from the United States Geological Survey (USGS) topographic maps or from other sources. An economic study should consider material costs, cost of foundations and erection, different structure heights, hardware costs, and right-of-way costs. The estimates are intended to give borrowers an idea as to relative rankings of various structure types and configurations such as steel lattice, steel pole, prestressed concrete pole, and wood H-frame or single pole. However, in the decision-making process, the utility may want to consider as part of the evaluation such intangibles as importance of the line to the power system, appearance, material availability, and susceptibility to environmental attack. In some areas, State or local constraints may ignore economics and specify the type of structure to be used.

The level ground span used to develop preliminary cost estimates in the economic study is determined from clearance-to-ground and structure strength. Developing a graph, as shown in Figure 1, is one means of determining the level ground span (points A and B).

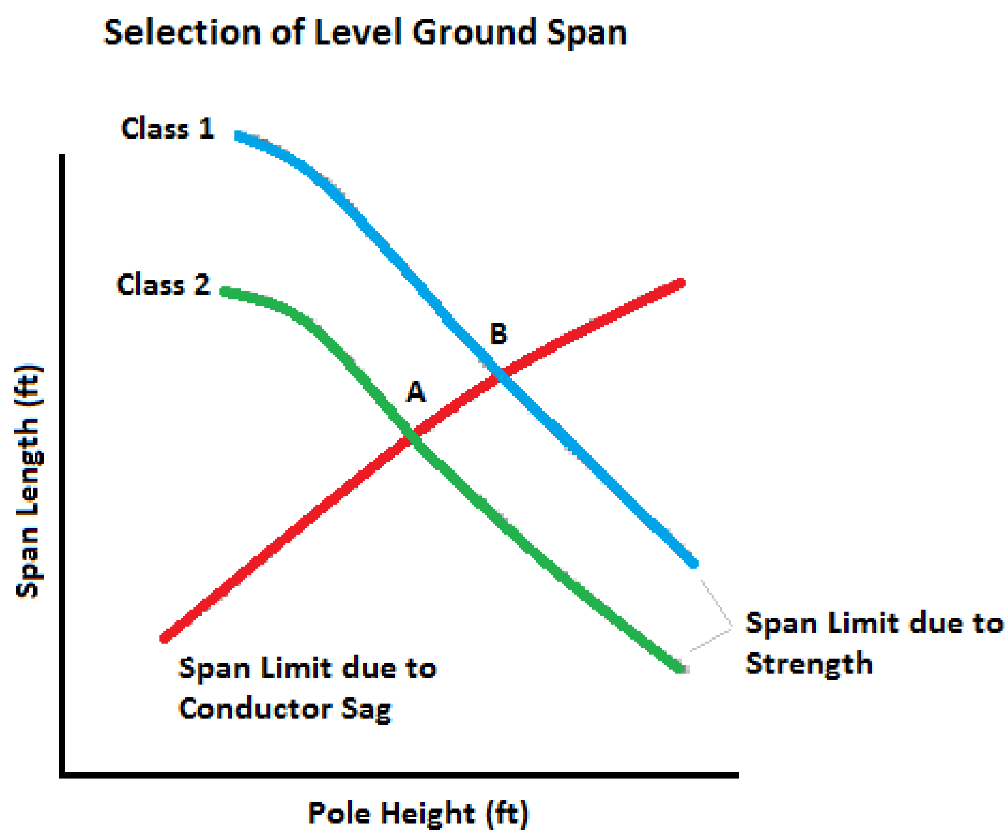

Figure 1

Cost per mile can be related to pole height and class of poles as shown in Figure 2. To keep the cost down, the line design should be based on one tangent structure type and one or two pole classes for the majority of the line. For H-frame structures, the engineer should consider double crossbraced structures, as well as single crossbraced structures.

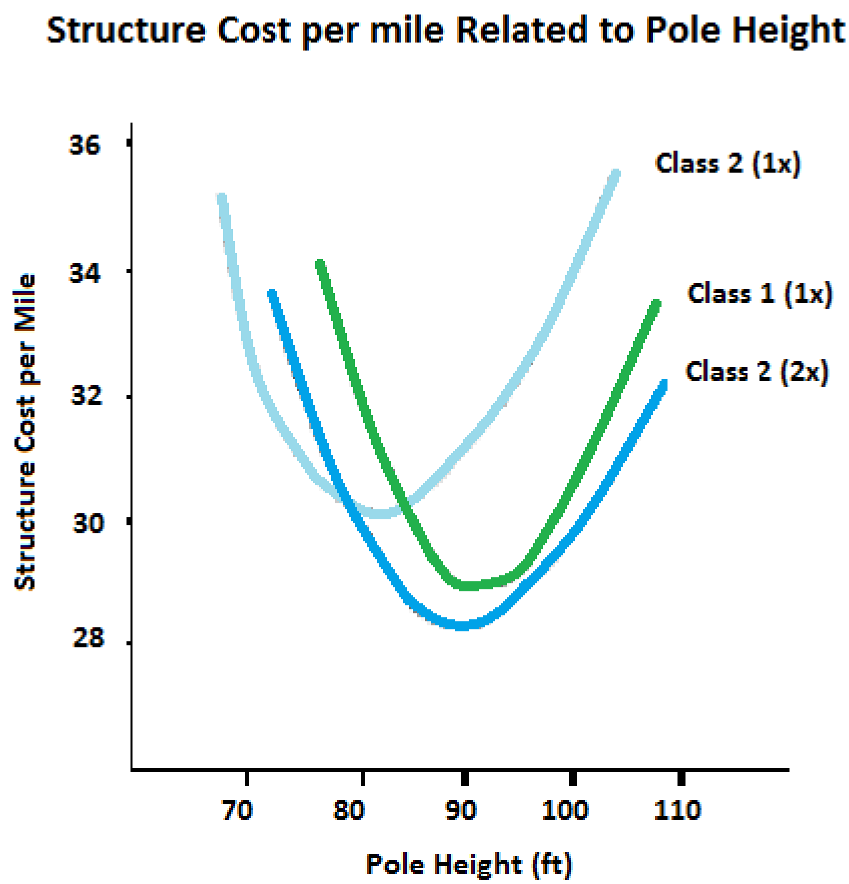

Figure 2

With the help of computer automated line design and optimization software, an economic study can be accomplished almost concurrently with the line design. If a land profile is available, or developed from USGS maps, the line designer may want to use optimization software to help determine the most economic line design. With such software, different structure types and materials and different conductor types can be evaluated. An advantage of optimization software is the use of the actual terrain (rather than level ground span) or a good approximation of the terrain. Optimization algorithms can fit structure height and type to the terrain, and can make use of different structure heights and configurations. The major disadvantage of optimization software is that it requires input and analysis of large amounts of data.

Wood Structures - General Design Considerations

The structural stress limitations set forth in Table 1 are recommended for transmission lines using standard wood pole construction. These values assume that the wood has not deteriorated due to decay occurring in the manufacturing process.

Table 1 Designated Stresses For Poles		
Kind of Wood	**Modulus of Elasticity * 1000 (psi)**	**Designated Ultimate Bending Stress (M.O.R.)* (psi)**
Western larch	1710	8400
Southern yellow pine	1800	8000
Douglas fir	1920	8000
Lodgepole pine	1340	6600
Jack pine	1220	6600
Red (Norway) pine	1630	6600
Ponderosa pine	1260	6000
Western red cedar	1120	6000
Northern white cedar	800	4000
*M.O.R. = Modulus of Rupture		

Douglas fir and Southern yellow pine (SYP) are used for crossarms. Southern yellow pine has four species which are long leaf (most popular species), loblolly, shortleaf, and slash. The coast type Douglas fir is the only type which should be used when specifying Douglas fir for crossarms. Table 2 gives strength properties to be used in crossarm design.

Table 2 Designated Stresses For Crossarms					
Kind of Wood	**Modulus of Elasticity x 1000 (psi)**	**Designated Ult. Bending Stress (M.O.R.)* (psi)**	**End Grain Max Crushing Strength (psi)**	**Across Grain Stress (psi)**	**Shear Parallel to Grain (psi)**
Douglas fir	1920	7400	7420	910	1140

SYP	1800	7400	7070	1000	1310
*M.O.R. = Modulus of Rupture					

The decay of poles results from fungi and other low forms of plant life which attack untreated poles or poles with insufficient preservative. Damage by insect attack (termites, ants, and wood borers) is also associated with decay. When preservative retention is low, wood cannot resist attacks by fungi and insects. There are two general classes of preservative treatment.

- Oil-Borne Using Creosote, Penta and Copper Naphthenate in Petroleum**.** Creosote oil was the predominant preservative for poles on rural systems until about 1947. Post-war shortages prompted the introduction of pentachlorophenol (penta) and copper naphthenate dissolved in the fuel oils, and other preservatives.

- Waterborne Using Arsenates of Copper**.** Poles using waterborne arsenates of copper (CCA, ACA and ACZA) are green in appearance. These preservatives were developed before World War II and have proven very effective as wood preservatives around the world.

Single pole wood structures are mainly limited in use to 115 kV and below. The six primary standard single pole structures utilized by most utilities include:

- Pin insulators
- Post insulators
- Pin or post insulators, double circuit
- Suspension insulators, crossarm construction
- Suspension insulators, crossarms, double circuit
- Suspension insulators, "wishbone" arm construction
- Suspension insulators, steel upswept arm construction

Design Calculations for Single Wood Pole Structures

Single pole wood structures shall be designed to withstand the loads multiplied by the appropriate load factors without exceeding the permitted stress level at the point of maximum stress. NESC Rule 261 states that for wood poles 55 ft or less in total length, design shall not exceed the permitted stress level at the ground line.

For poles 60 ft or longer in total length, the point of maximum stress shall be determined according to ANSI 05.1.2008. Typically for poles 60 ft or longer, the point of maximum stress occurs above the ground line. According to ANSI 05.1.2008, the theoretical point of maximum stress for a single pole with a uniform taper is located where the circumference is one and one half times the circumference at the point of applied, or net pull, load. At the point of maximum stress, the resulting available moment is to be multiplied by the appropriate strength factors.

For determining maximum horizontal spans limited by pole strength, two methods are to be used.

- The Ground Line Method shall be used for poles 55 ft or less in total length.
- For poles 60 ft or longer in total length, the Point of Maximum Stress Method shall be used. Example problems follow that illustrate how these methods are to be used.

Maximum Horizontal Span Limits of Single Wood Pole Structures

The following conditions should be taken into account when determining horizontal spans as limited by pole strength for tangent structures:

- Wind on the conductors and OHGW is the primary load. 75 to 90 percent of the horizontal span will be determined by this load.
- Wind on the structure will affect the horizontal span by 5 to 15 percent.
- Unbalanced vertical load will increase ground line moments. For single circuit structures, one phase is usually left unbalanced. The vertical load from the conductor will induce moments at the ground line or point of maximum stress and will affect horizontal span lengths by 2 to 10 percent.
- P-delta (P-δ) moments will also increase induced moments. As a transverse load is applied to a structure, the structure will deflect. This deflection will offset the vertical load an additional amount " δ " causing an additional moment of the vertical weight times this deflection. This additional moment due to deflection is a secondary effect.

The strength of the crossarm for Tangent Suspension structures and the strength of the insulator for Tangent Post structures have to be checked to determine its ability to withstand all expected vertical and longitudinal loads. When determining bending stress in crossarms, moments are calculated at the through bolt, without considering the strength of the brace. The vertical force is determined by the vertical span under those conditions which yield the maximum vertical weight. The strength of two crossarms will be twice the strength of one crossarm. When considering the strength of the crossarm to withstand longitudinal loadings, reduction in the moment capacity due to bolt holes should be taken into account.

The following equation is the general equation for determining the moment induced in the pole from the applied loads represented in Figures 3 and 4. This equation is the basis for the Ground Line and Point of Maximum Stress Methods to be used to determine maximum horizontal spans.

$$\Phi * M_A = M_g = (LF * M_{wp}) + (LF * M_{wc}) + (LF * M_{vo}) + LF * M_{p-\delta}$$

Where,
ϕ = strength factor
M_A = $F_{b*}S$, the ultimate moment capacity of the pole, ft-lbs.
F_b = designated ultimate bending stress (M.O.R.)
S = section modulus of the pole
LF = load factor associated with the particular load
M_g = the induced moment
M_{wc} = groundline moment due to wind on the wires
M_{wp} = groundline moment due to wind on the pole
M_{vo} = groundline moment due to unbalanced vertical load
$M_{p\text{-}\delta}$ = groundline moment due to pole deflection

When estimating the load carrying capacity of a pole using manual methods, it is difficult to assess the additional moment due to deflection. Because moment due to deflection, $M_{p\text{-}\delta}$, is a function of the *vertical span* (VS), the engineer should make an assumption about the relationship between the vertical and *horizontal span* (HS). The relationship used is:

VS = 1.25*HS

Ground Line Method for Poles 55 Feet and Under

Refer to Figures 3 or 4 when considering the equations and symbols that follow.

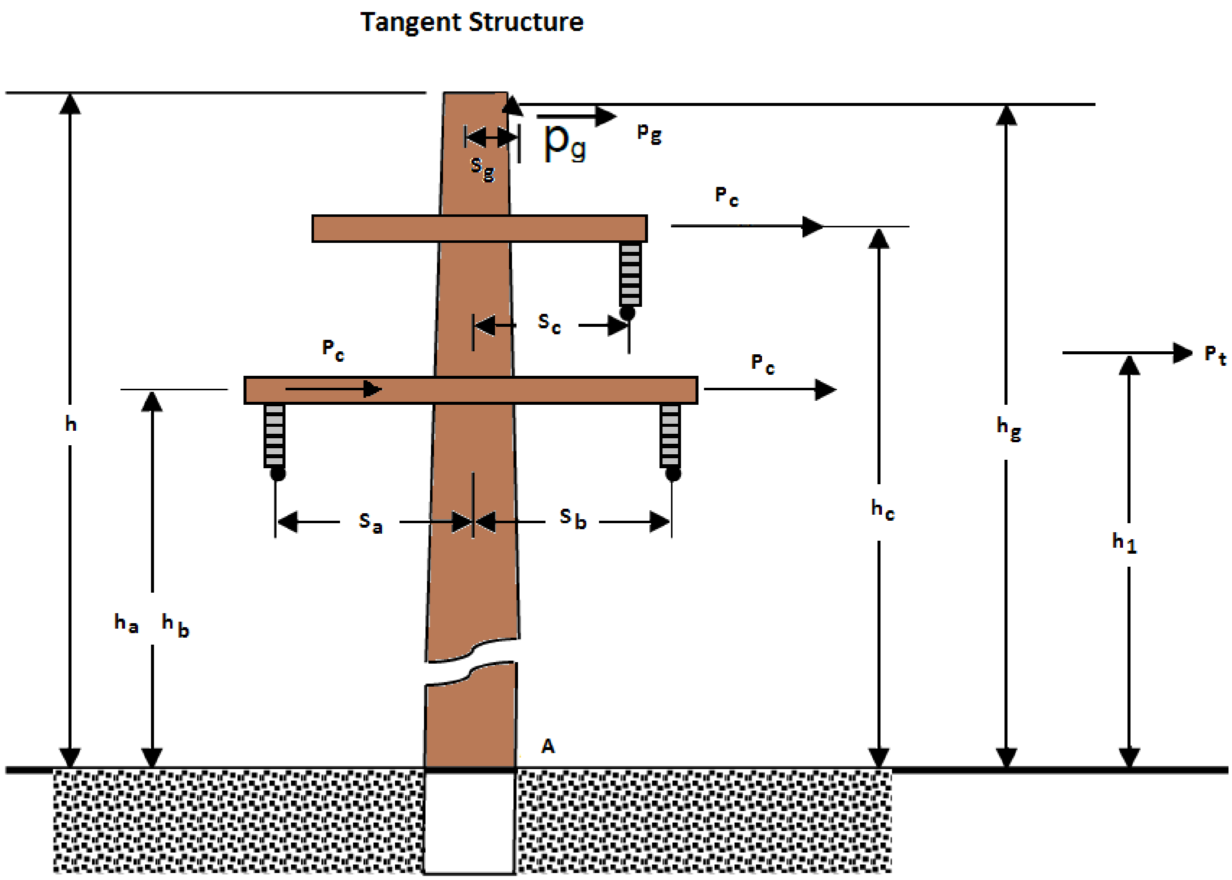

Figure 3

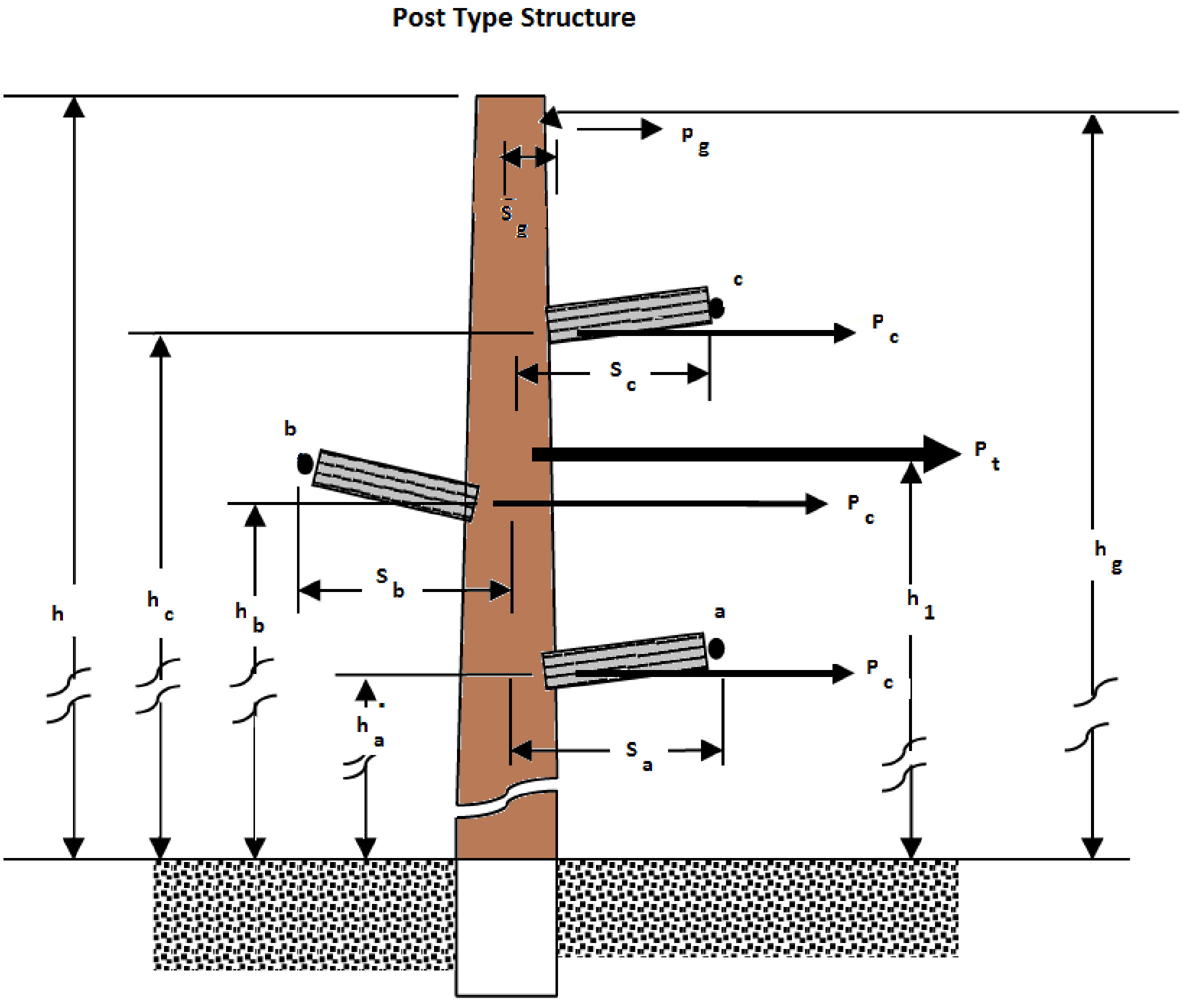

Figure 4

Step 1.

M_{wp} = groundline moment due to wind on the pole

$$\mathbf{M_{wp}} = \frac{\mathbf{F * (2d_t + d_a) * h^2}}{\mathbf{72}}$$

Where,

F = wind pressure, PSF

d_t = diameter of pole at top, inches

d_a = diameter of pole at groundline, inches

h = height of pole above groundline, feet

Step 2.

M_{wc} = groundline moment due to wind on the wires

$$\mathbf{M_{wc} = P_t * h_1 * HS}$$

Where,

HS = horizontal span, feet

h_1 = arm of pt, feet

$$\mathbf{h_1 = \frac{(h_a * p_c) + (h_b * p_c) + (h_c * p_c) + (h_g * p_g)}{P_t}}$$

p_c, p_g = transverse wire loads

P_t = sum of transverse unit wire loads, lbs/ft; in example,

$P_t = 3\ p_c + p_g$ for single circuit, single pole structures

Step 3.

M_{vo} = groundline moment due to unbalanced vertical load

$$\mathbf{M_{vo} = 1.25 * HS * (w_c s_t + w_g s_g) + (W_i * S_t)}$$

Where,

s_g = Horizontal distance from center of pole to ground wire (positive value on one side of the pole, negative on the other), feet

$s_t = s_a + s_b + s_c$,

s_a , s_b ,and s_c = horizontal distances from center of pole to conductors (positive value on one side of the pole, negative on the other), feet

w_c = weight of the conductor per unit length, lbs./ft.

w_g = weight of overhead groundwire per unit length, lbs./ft.

W_i = weight of insulator, lbs. (For Tangent Post structure, $W_i = 0$)

Step 4.

$M_{p\text{-}\delta}$ = groundline moment due to pole deflection

$$\mathbf{M_{p\text{-}\delta} = 1.25 * HS * W_t * \delta_{imp}}$$

Where,

w_t = total weight per unit length of all wires, lbs./ft.

HS = Horizontal span, ft

Step 5.

δ_{imp} = improved estimate of deflection of the structure, ft.

$$\boldsymbol{\delta_{imp} = \frac{6.78 * p_t * HS * h_1^3 * 144}{E * d_g^3 * d_1} * \delta_{magp}}$$

Where,
E = modulus of elasticity, psi
d_g = diameter of pole at groundline, inches
d_1 = diameter of pole, inches, at height "h_1" above ground line
δ_{mag} = deflection magnifier, no units, (assume 1.15 initially)
h_1 = effective height to the conductors, feet
HS = horizontal span, feet
p_t = total transverse load per unit length of all wires, lbs./ft.

Step 6.
After substitutions of M_{wp}, M_{wc}, M_{vo}, and $M_{p\delta}$ have been made into the equation it can be reduced to a quadratic equation and solved for the horizontal span.

$$\boldsymbol{\Phi * M_A = M_g = (LF * M_{wp}) + (LF * M_{wc}) + (LF * M_{vo}) + LF * M_{p-\delta}}$$

Using the relationship,

a(HS)2 + b(HS) + c =0

We can find HS as,

$$\boldsymbol{HS = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}}$$

Step 7.
Once "HS" has been calculated, check the assumption of δ_{mag} = 1.15:

$$\boldsymbol{\delta_{mag} = \frac{1}{1 - \frac{1.25 * HS * W_t}{P_{cr}}}}$$

(See example problems for calculations of P_{cr})

Point of Maximum Stress Method for Poles 60 feet and longer

For wood poles 60 feet or longer in total length and utilizing either the Tangent Suspension or Tangent Post configuration, the following procedure should be followed to calculate the *point of maximum stress*, P_{OMS}, on the pole and resulting allowable moment and maximum horizontal span.

For wood poles 60 feet, or longer, in total length, utilizing either the Tangent Post or Tangent Suspension pole type assembly, refer to Figures 5 and 6.

Tangent Structure, P_{oms}

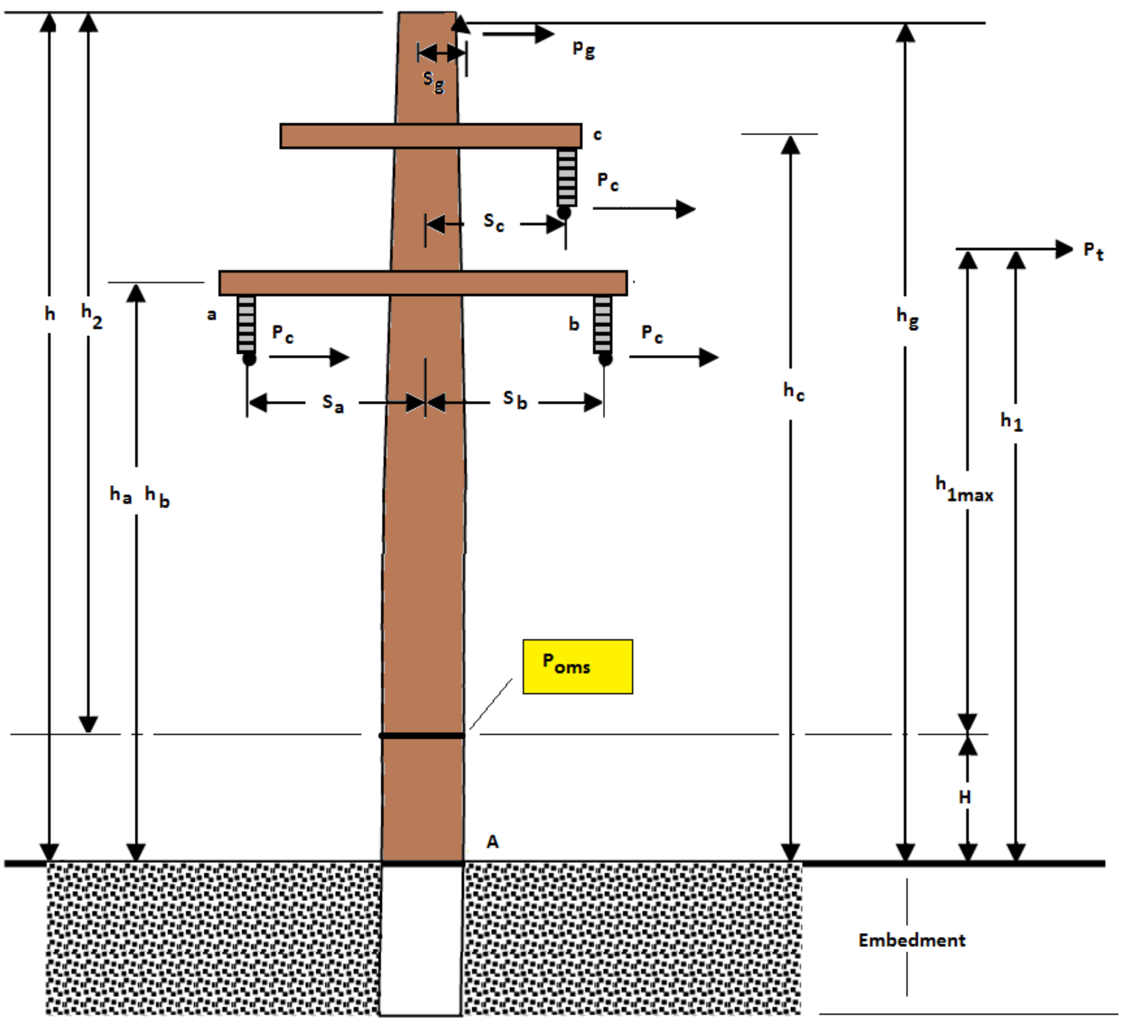

Figure 5

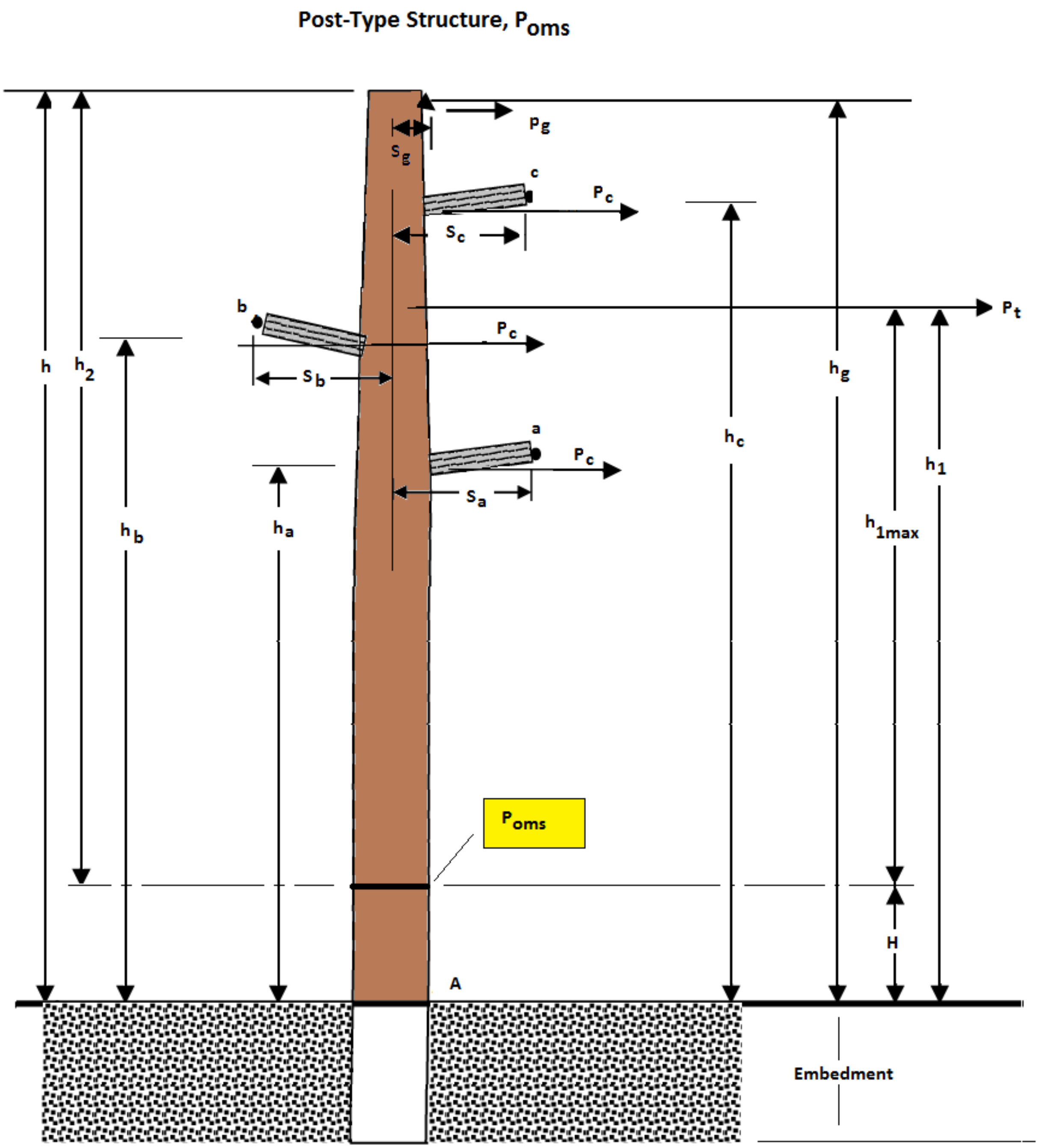

Figure 6

Step 1.
Calculate location of the resultant load (net pull), Pt

$$\mathbf{h_1} = \frac{(\mathbf{h_a} * \mathbf{p_c}) + (\mathbf{h_b} * \mathbf{p_c}) + (\mathbf{h_c} * \mathbf{p_c}) + (\mathbf{h_g} * \mathbf{p_g})}{\mathbf{P_t}}$$

Where,
h_1 = Moment arm of P_t above groundline, feet
p_c, p_g = transverse wire loads
P_t = sum of transverse unit wire loads, lbs/ft;
$P_t = 3\ p_c + p_g$ for single circuit, single pole structures

Step 2.
Find the circumference, C_{ms}, at the point of maximum stress, P_{oms}.

C_{ms} = 1.5 times circumference at P_t

Calculate distance from "Pt" to top of pole

Using appropriate pole manufacturer's data, determine diameter of pole at P_t, d_{pt}. Interpolate if necessary.

Calculate the circumference at "pt", C_{pt}. Use the pole diameter, at "pt" (D_{pt}):

$$\mathbf{C_{pt} = \pi * d_{pt}}$$

Multiply C_{pt} by 1.5 to calculate C_{ms} :

$$\mathbf{C_{ms} = 1.5 * C_{pt}}$$

Step 3.
Locate where P_{oms} occurs on the pole.

Calculate the pole diameter, d_{ms}, at the point of maximum stress,

$$\mathbf{d_{ms}} = \frac{\mathbf{C_{ms}}}{\boldsymbol{\pi}}$$

Determine where d_{ms} occurs on the pole. Interpolate if necessary. This determines distance, h_2, from top of pole to d_{ms} and is the location of the P_{oms}.

Subtract h_2 from pole height above ground, h, to determine distance, H, from groundline to P_{oms}

$h_{1max} = h_1 - H$

Step 4.

Calculate the allowable moment, Ma, at the point of maximum stress of the pole.

$$\boldsymbol{M_a = F_b * S}$$

$$\mathbf{S = \frac{\pi * d_{ms}^3}{32}}$$

Where,

F_b = fiber stress, psi

S = Section Modulus at point of maximum stress, in^3

Step 5.

Calculate the maximum horizontal span based on the point of maximum stress of the pole.

Moment at P_{oms} due to wind on pole:

$$\mathbf{M_{wp} = \frac{F * (2d_t + d_{ms}) * \boldsymbol{h_2^2}}{72}}$$

Where,

h_2 = distance from P_{oms} location to top of pole, feet

d_t = pole diameter at top, inches

d_{ms} = pole diameter at $P_{oms,}$ inches

F = wind pressure, PSF

Moment at P_{oms} due to wind on wires:

$$\mathbf{M_{wc} = P_t * h_{1max} * HS}$$

Moment at P_{oms} due to unbalanced vertical load:

$$\mathbf{M_{vo} = 1.25 * HS * (w_c s_t + w_g s_g) + (W_i * S_t)}$$

Note: For Tangent Post structure, $W_i = 0$

Moment at P_{oms} due to pole deflection:

$$\mathbf{M_p\text{-}\delta = 1.25 * HS * W_t * \delta_{imp}}$$

Where,
w_t = total weight per unit length of all wires, lbs./ft.
δ_{imp} = improved estimate of deflection of the structure, ft.

$$\boldsymbol{\delta_{imp} = \frac{6.78 * p_t * HS * h_1^3 * 144}{E * d_g^3 * d_1} * \delta_{imp}}$$

E = modulus of elasticity, psi
d_{ms} = diameter of pole at point of maximum stress, inches
d_{pt} = diameter of pole at the resultant load, p_t, inches
δ_{mag} = deflection magnifier, no units, (assume 1.15 initially)
h_{1max} = distance from location of P_t to P_{oms}, feet
HS = horizontal span, feet
P_t = total transverse load per unit length of all wires, lbs./ft.
s_g = Horizontal distance from center of pole to ground wire
s_c = horizontal distances from center of pole to conductors
w_c = weight of the conductor per unit length, lbs./ft.
w_g = weight of overhead groundwire per unit length, lbs./ft.

After substitutions of M_{wp}, M_{wc}, M_{vo}, and $M_{p-\delta}$ have been made into the equation it can be reduced to a quadratic equation (below) and solved for the horizontal span.

$$\mathbf{\Phi * M_A = M_g = (LF * M_{wp}) + (LF * M_{wc}) + (LF * M_{vo}) + LF * M_{p-\delta}}$$

$$\mathbf{HS = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}}$$

Once "HS" has been calculated, check the assumption of $\delta_{mag} = 1.15$:

$$\boldsymbol{\delta_{imp} = \frac{1}{1 - \frac{1.25 * HS * W_t}{P_{cr}}}}$$

Note: See example problems for calculations of P_{cr}.

The following example problems illustrate calculating the maximum horizontal span using the Ground Line and Point of Maximum Stress Methods.

Example Problem for Maximum Horizontal Span for Tangent Structure

Using the Ground Line Method, determine the maximum horizontal span for a 55 ft, Class 1, Southern Yellow Pine, 69 kV tangent suspension wood structure (Figure 5). Terrain is predominantly level, flat, and open. ("s_g" is negligible). Location and magnitude of resultant loads are indicated in Figure 7.

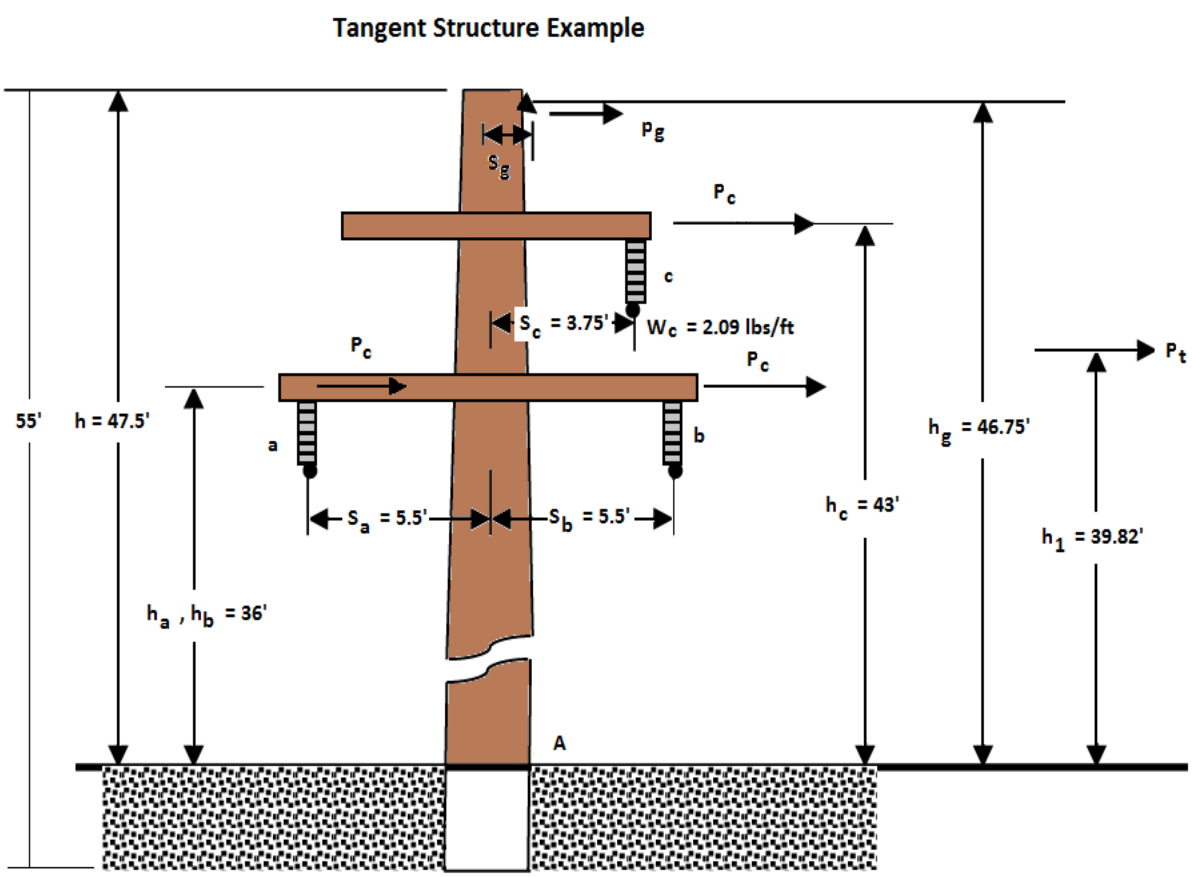

Figure 7

Given:

NESC Heavy Loading: 4 PSF
0.5 inch radial ice

Load Factors:

Transverse Wind: 2.5
Vertical Loads: 1.5

Extreme wind: 19 PSF on the wires
22 PSF on the structure

Extreme Ice with Concurrent Wind (EI&W): 4 PSF, 1 inch radial ice
Pole: 55-1, Southern Yellow Pine
Conductor: 795 kcmil, 26/7 ACSR (Drake)
Ground wire: 3/8" H.S.S.

Conductor loads, lbs./ft:

	Heavy	Extreme Wind	EI&W
Transverse	0.7027	1.754	1.036
Vertical	2.0938	1.0940	3.7154

Ground wire loads, lbs./ft:

	Heavy	Extreme Wind	EI&W
Transverse	0.4533	0.5700	0.7868
Vertical	0.8079	0.2730	1.9667

Other information:
Fb(pole) = 8000 psi
Fb (crossarm) = 7400 psi
Wi , Wt. of insulator = 58 lbs.
Dia. (top) = 8.59 in.
Dia. (groundline) = 14.61 in.
H2 = 55' – (55*10% +2) = 47.4 ft.

p_t(total unit load) + wt(total unit load) = 3(0.7027) + 0.4533 = 2.5614 lbs./ft.
W_t (total unit load) = 3(2.0938) + 0.8077= 7.0891 lbs./ft
P_{cr}(critical buckling load) = 25,098 lb
S_g= 0.5'

$$h_1 = \frac{(h_a * p_c) + (h_b * p_c) + (h_c * p_c) + (h_g * p_{g)}}{P_t}$$

$$h_1 = \frac{(36 * 0.7027) + (36 * 0.7027) + (43 * 0.7027) + (46.75 * 0.4533)}{2.5614}$$

h_1 = 39.82 ft = 478 in

For the critical buckling load, P_{cr},

E = $1.8*10^6$ psi
d_a = d_{pt} = Diameter at h_1 above ground line= 9.57 in

d_g = Diameter at ground line = 14.61 in

By interpolating the diameter at h_1 (39.82') is found by

$$d_{pt} = 14.61 - 39.82 * \frac{(14.61 - 8.59)}{47.5} = 9.57''$$

$$\mathbf{I = \frac{\pi * d_{pt}^4}{64}}$$

I = 412 in^4

$$\mathbf{P_{cr} = \frac{\pi^2 * E * I}{4 * l^2} * (\frac{d_g}{d_a})^{2.7}}$$

$$P_{cr} = \frac{\pi^2 * 1.8 * 10^6 * 412}{4 * 478^2} * (\frac{14.61}{9.57})^{2.7}$$

P_{cr} = 25,098 lb

Maximum Horizontal Span Considering P-δ moments

A comparison of unit loads with load factors indicates that the Heavy Loading District Loads control design. Therefore, for Heavy Loading, the moments are calculated as follows,

Step1.

Moments due to wind on the pole,

$$M_{wp} = \frac{F * (2d_t + d_a) * h^2}{72}$$

$$M_{wp} = \frac{4 * (2 * 8.59 + 14.61) * 47.5^2}{72}$$

M_{wp} = 3,985 ft –lb

Step 2.

Moments due to wind on the conductors,

$M_{wc} = P_t * h_1 * HS$

$M_{wc} = 2.5614 * 39.82 * HS$

$M_{wc} = 102.0 * HS$

Step 3.
Moments due to unbalanced vertical load,

$M_{vo} = 1.25 * HS * (w_c s_c + w_g s_g) + (W_i * S_c)$

$M_{vo} = 1.25 * HS * (2.0938 * 3.75 + 0.8077 * 0.5) + (58 * 3.75)$

$M_{vo} = 10.31HS + 217$

Step 4.
Groundline moments due to deflection,

$M_{p\text{-}\delta} = 1.25 * HS * W_t * \delta_{imp}$

$M_{p\text{-}\delta} = 1.25 * HS * 7.0891 * \delta_{imp}$

$$\delta_{imp} = \frac{6.78 * p_t * HS * h_1^3 * 144}{E * d_g^3 * d_1} * \delta_{imp}$$

assume δ_{mag} = 1.15 initially

$$\delta_{imp} = \frac{6.78 * 2.5614 * HS * 39.8^3 * 144}{1.8 * 10^6 * 14.61^3 * 9.57} * 1.15$$

$\delta_{imp} = 0.0033HS$

$M_{p\text{-}\delta} = (1.25*HS) * (7.0891* 0.0033*HS)$

$M_{p\text{-}\delta} = 0.0292HS^2$

Step 5.

$$\Phi * M_A = (LF * M_{wp}) + (LF * M_{wc}) + (LF * M_{vo}) + LF * M_{p-\delta}$$

From pole manufacturer's data, M_A = 204.2 ft-K groundline moment at embedment = 7.5 ft,
Therefore,

0.65 * 204,200 = (2.5 * 3,985) + (2.5 * 102HS) + (1.5 *10.31HS + 217) +(1.5 * 0.0292HS^2)

132,730 = 9,963+ 255HS +15.465HS + 325.5+ 0.0438HS^2

0.0438HS^2 + 270.465HS −122,442 = 0

Step 6.

a(HS)^2 + b(HS) + c = 0

$$\mathrm{HS} = \frac{-\mathrm{b} \pm \sqrt{\mathrm{b}^2 - 4\mathrm{ac}}}{2\mathrm{a}}$$

$$\mathrm{HS} = \frac{-270.465 \pm \sqrt{270.465^2 - 4 * 0.0438 * -122{,}422}}{2 * 0.0438}$$

HS = 423 feet

Step 7.

Once the HS has been calculated, the assumption of 1.15 as the magnifier should be checked.

P_{cr} = 25,098 lb assuming fixed free conditions.

$$\delta_{\mathrm{imp}} = \frac{1}{1 - \frac{1.25 * HS * W_t}{P_{cr}}}$$

$$\delta_{\mathrm{imp}} = \frac{1}{1 - \frac{1.25 * 423 * 7.0891}{25{,}098}}$$

δmag = 1.17

Since the δ_{mag} check shows a value different from 1.15, recalculate the HS using 1.17 as the δ_{mag}. Recalculating the HS with δ_{mag} = 1.17 gives HS = 423 feet. Assumption of 1.15 as δ_{mag} is acceptable.

Step 8.
The Equivalent load two feet from the top is approximately 4,300 lbs. For average soil, the embedment depth for a 4300 lb. load 2 feet from the top is between 7.5 and 8.5 feet (See Figure 2, Volume IV). Lines nearby have performed well with the standard embedment depths. Engineering judgment dictates that a 7.5 foot embedment depth for the 55 foot pole will be sufficient.

Example Problem for Maximum Horizontal Spans for Tangent Post Structure

Using the Point of Maximum Stress Method**,** determine the maximum horizontal span for a 69 kV post insulator structure, 70 ft, Class 1, Southern Yellow Pine pole (Figure 8). Terrain is predominantly level, flat, and open. ("s_g" is negligible). Location and magnitude of resultant loads are indicated in Figure 8. Similar calculations can be performed for Tangent Suspension pole top assemblies.

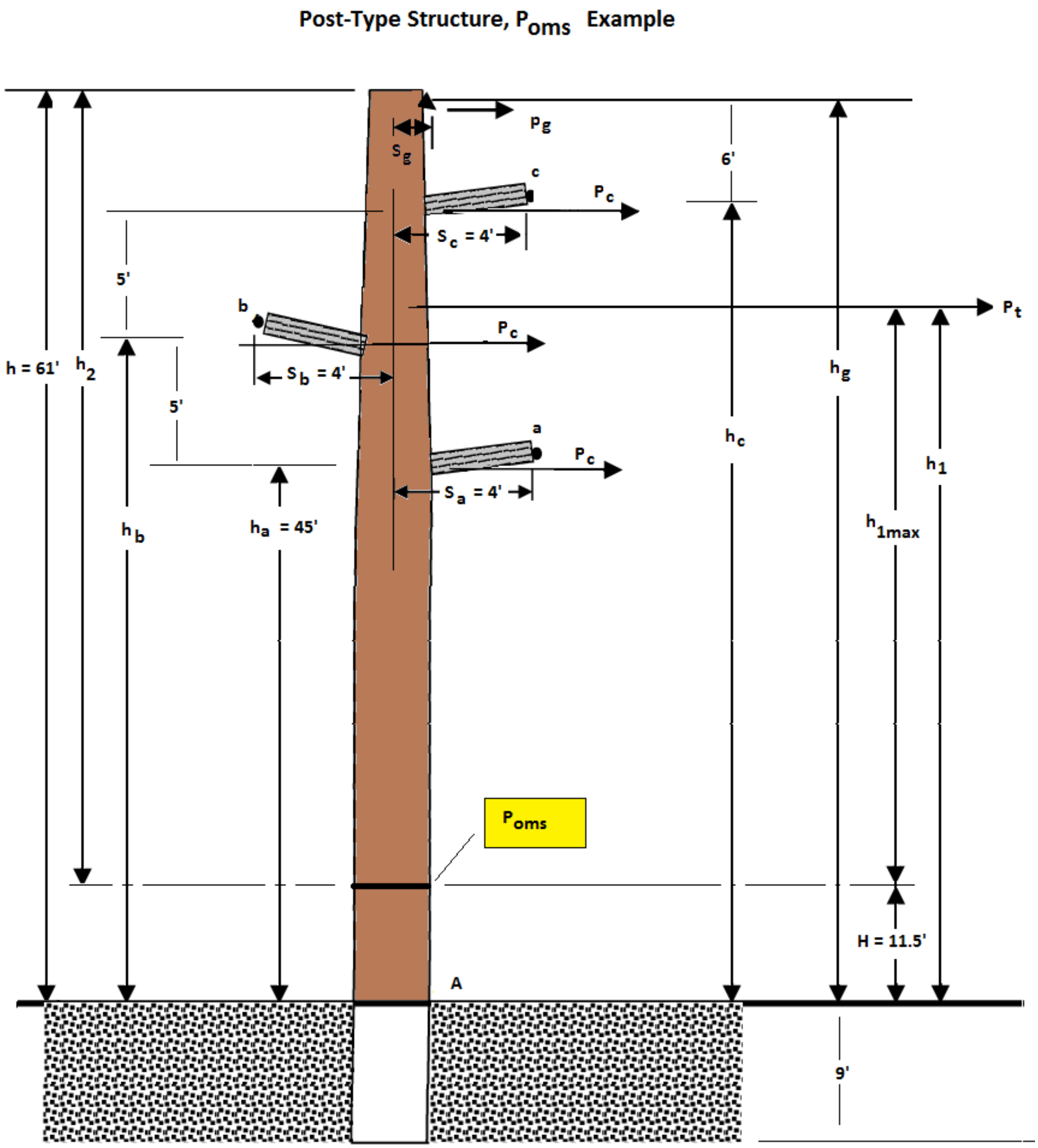

Figure 8

<u>Given</u>:

NESC Heavy Loading: 4 PSF, 0.5 inch radial ice

Extreme wind: 19 PSF on the wires

22 PSF on the structure

Extreme Ice with 4 PSF, 1 inch radial ice

Concurrent Wind (EI&W)

Pole: Southern Yellow Pine
Conductor: 795 kcmil, 26/7 ACSR (Drake)
Ground wire: 3/8" H.S.S.

Conductor Loads:

	Heavy	Extreme Wind	EI&W
Transverse	0.7027	1.754	1.036
Vertical	2.0938	1.0940	3.7154

Ground wire loads, lbs./ft**:**

	Heavy	Extreme Wind	EI&W
Transverse	0.4533	0.5700	0.7868
Vertical	0.8079	0.2730	1.9667

Other information:
F_b =8,000 psi
Diameter (top of pole) =8.594 in
Diameter (groundline) =15.88 in
p_t (total wire transverse unit load) =2.5614 lbs./ft.
w_t (total wire vertical unit load) = 7.891 .lbs/ft

Step 1.
Calculate location of the resultant load, P_t

P_t = 3 * 0.7027 + 0.453 = 2.5614 lb/ft

$$h_1 = \frac{(45 * 0.7027) + (50 * .07027) + (55 * 0.7027) + (60.5 * 0.4533)}{2.5614}$$

h_1 = 51.9 ft. above ground line

Step 2.
Find the circumference, C_{ms}, at the point of maximum stress

P_t at 60.9 feet from the pole butt (51.9 ft + 9 ft embedment), or 9.1 feet from pole top. From pole data, for a 70/1 wood pole, interpolate between diameter at 9 feet and 10 feet from pole top to get diameter at P_t .

d_{pt} = 9.66 inches
Pole circumference at P_t,

$C_{pt} = \pi * 9.66$ in $= 30.35$ in
$C_{ms} = 1.5 * 30.35$ in
$C_{ms} = 45.53$ in

Step 3.
Locate where the point of maximum stress, P_{oms}, occurs on the pole.

$$d_{ms} = \frac{C_{ms}}{\pi} = 14.50 \text{ in.}$$

For the 70 foot, class 1 pole, a diameter of 14.5 inches occurs 49.5 ft from the top of the pole by interpolation. Therefore, the P_{oms} occurs 49.5 feet from the top of the pole, which is 11.5 feet above the ground line.

$P_{oms} = 49.5$ feet (from tip)
$P_{oms} = 11.5$ feet (above ground line)

Step 4.
Calculate the allowable moment, M_a, at P_{oms}

$F_b = 8{,}000$ psi

$$S = \frac{\pi * 14.5^3}{32} = 299.3 \text{ in}^3$$

$M_a = F_b * S = 8000 * 299.3 = 199{,}533$ ft-lbs

Step 5.
Calculate the maximum horizontal span based on point of maximum stress of the pole

Moment at P_{oms} due to wind on Pole:

$$M_{wp} = \frac{4 * (2 * 8.59 + 14.5) * 49.5^2}{72}$$

$M_{wp} = 4{,}312$ ft-lb

Moment at P_{oms} due to wind on wires:

$M_{wc} = P_t * h_{1\,max} * HS$

$M_{wc} = 2.5614 * 40.4 * HS$

$M_{wc} = 103.5\ HS$

Moment at P_{oms} due to vertical offset:

$M_{vo} = 1.25HS * (w_c * s_c + w_g * s_g) + W_i * S_t$

$M_{vo} = 1.25HS * (2.0938 * 4 + 0.807 * 0.5)$

$M_{vo} = 11.0HS$

Secondary Moments:

$$\delta_{imp} = \frac{\mathbf{6.78 * p_t * HS * h_{1\,max}^3 * 144}}{\mathbf{E * d_{ms}^3 * d_{pt}}} * \boldsymbol{\delta_{mag}}$$

Where,
Pt = 2.5614 lb/ft
d_{pt} = 9.7 in
d_{ms} = diameter at P_{oms} = 14.5 in
δ_{mag} = assume 1.15
h_{1max} = 40.4 ft
E = 1.8 x 10^6 psi

$$\delta_{imp} = \frac{6.78 * 2.5614 * HS * 40.4^3 * 144}{1.8 * 10^6 * 14.5^3 * 9.7} * 1.15$$

$$\delta_{imp} = 0.0035\ HS$$

$\mathbf{M_p\delta = 1.25 * HS * W_t * \delta_{imp}}$

$M_p\delta = 1.25 * HS * 7.0891 * 0.0035\ HS$

$M_p\delta = 0.0310\ HS^2$

Total Moments:

$$\Phi * M_A = (LF * M_{wp}) + (LF * M_{wc}) + (LF * M_{vo}) + LF * M_{p\delta}$$

$$0.65 * 199{,}533 = 2.5 * 4312 + 2.5 * 103.7\ HS + 1.5*11\ HS + 1.5 * 0.0310\ HS^2$$

$$0.0465\ HS^2 + 275.8\ HS - 118{,}916 = 0$$

$$HS = \frac{-275.8 \pm \sqrt{275.8^2 - 4 * 0.0465 * -118{,}916}}{2 * 0.0465}$$

HS = 403 ft

Check Magnifier:

$$\delta_{imp} = \frac{1}{1 - \frac{1.25 * HS * W_t}{P_{cr}}}$$

Where,

$$P_{cr} = \frac{\pi^2 * E * I}{4 * l^2} * \left(\frac{d_{max}}{d_{pt}}\right)^{2.7}$$

$$P_{cr} = \frac{\pi^2 * 1.8 * 10^6 * 434.6}{4 * 485^2} * \left(\frac{14.5}{9.7}\right)^{2.7}$$

P_{cr} = 24,296 lbs

d_{pt} = 9.7 in
d_{ms} = 14.5 in

$$I = \frac{\pi d_{pt}^4}{64}$$

$$I = \frac{\pi * 9.4^4}{64} = 434.6\ in^4$$

$l = h_{1max}$ = 40.4 ft = 485 in

$$\delta_{\text{imp}} = \frac{1}{1 - \frac{1.25 * 7.0891 * 403}{24{,}296}}$$

$\delta_{\text{imp}} = 1.172$, okay.

Note: If magnifier check had shown δ_{mag} to be significantly different from 1.15 on the order of ± 0.03 or greater, then recalculate the horizontal span, HS, using the new δ_{mag} that was found from the check.

Maximum Vertical Span for Tangent Post and Tangent Suspension Pole Top Assemblies

To determine the vertical span, the moment capacity of the arm at the pole is calculated.

Calculations for these structures are:

$$\mathbf{VS} = \frac{\boldsymbol{\phi} * \mathbf{M_{x-arm}} - \mathbf{LF} * \mathbf{W_i} * \mathbf{s_c}}{\mathbf{LF} * \mathbf{w_c} * \mathbf{s_c}}$$

Where,
$M_{x\text{-arm}}$ = Fb*S, moment capacity of the arm, ft-lbs
Fb = Tthe designated bending stress
S = The section modulus of the arm
w_c = Weight of the conductor per unit length, lbs./ft
s_c = Moment arm, feet
W_i = Insulator weight, 50 lbs.
VS = Vertical span, feet
φ = Strength factor

Example of Vertical Span Calculations for Tangent Suspension Pole Top Assembly

w_C =2.0938 lbs./ft.
S = 22.7 in^3 for a 4-5/8” x 5-5/8” crossarm
ϕ = 0.50
LF = 1.5 Heavy Loading District
Use 795 kcmil, 26/7 conductor

$$\text{VS} = \frac{0.50 * \text{M}_\text{a} - \text{LF} * \text{W}_\text{i} * \text{s}_\text{c}}{\text{LF} * \text{w}_\text{c} * \text{s}_\text{c}}$$

$M_a = F_b * S$

M_a = 7400 * 22.7 / 12 =14,000 ft-lbs

W_i = 50 lbs.

$$VS = \frac{0.50 * 14{,}000 - 1.5 * 50 * 5.5}{1.5 * 2.0938 * 5.5}$$

VS = 381 ft.

Check vertical span for extreme ice with concurrent wind**,**

Where,

w_c = 3.7154 lbs./ft.,

ϕ = 0.65 and

LF = 1.1 for extreme ice with concurrent wind

$$VS = \frac{0.65 * 14{,}000 - 1.1 * 50 * 5.5}{1.1 * 3.7154 * 5.5}$$

VS = 391 ft.

Example of Vertical Span Calculations for Tangent Post Pole Top Assembly

To determine the vertical span for Tangent Post pole top assembly, the moment capacity of the post insulator at the pole is calculated. For purposes of this calculation, a polymer post insulator was chosen with an ultimate cantilever strength of 6,060 lbs and a section length of 4 feet.

The weights per unit for the two conditions of the 795 kcmil, 26/7 ACSR "Drake" conductor are:

Heavy Loading District of 0.5 inch radial ice = 2.0938 lb-ft
Extreme radial ice of 1.0 inch = 3.7154 lb-ft

Span Limits for Heavy District Loading:

Utilize 40% of ultimate cantilever rating.

40% * 6,606 lbs = 2,424 lbs

2,424 lbs / 2.0938 lbs/ft =1,157 ft

Span limits for extreme ice condition:

In accordance with Table 6 in Volume III says to utilize 50% of ultimate cantilever rating.

0.50 * 6,606 lbs = 3,303 lbs

3,303 lbs / 3.7154 lbs/ft = 889 ft

The maximum vertical Span for this Tangent Post pole top assembly example is 889 ft.

Span Calculations for Wishbone-type crossarm assembly

The wishbone-type crossarm assembly is intended for use on transmission lines where conductor jumping due to ice unloading and/or conductor galloping are problems. The wishbone provides additional vertical and horizontal offset between phases in order to reduce the possibilities of phase-to-phase faulting due to ice unloading or galloping.

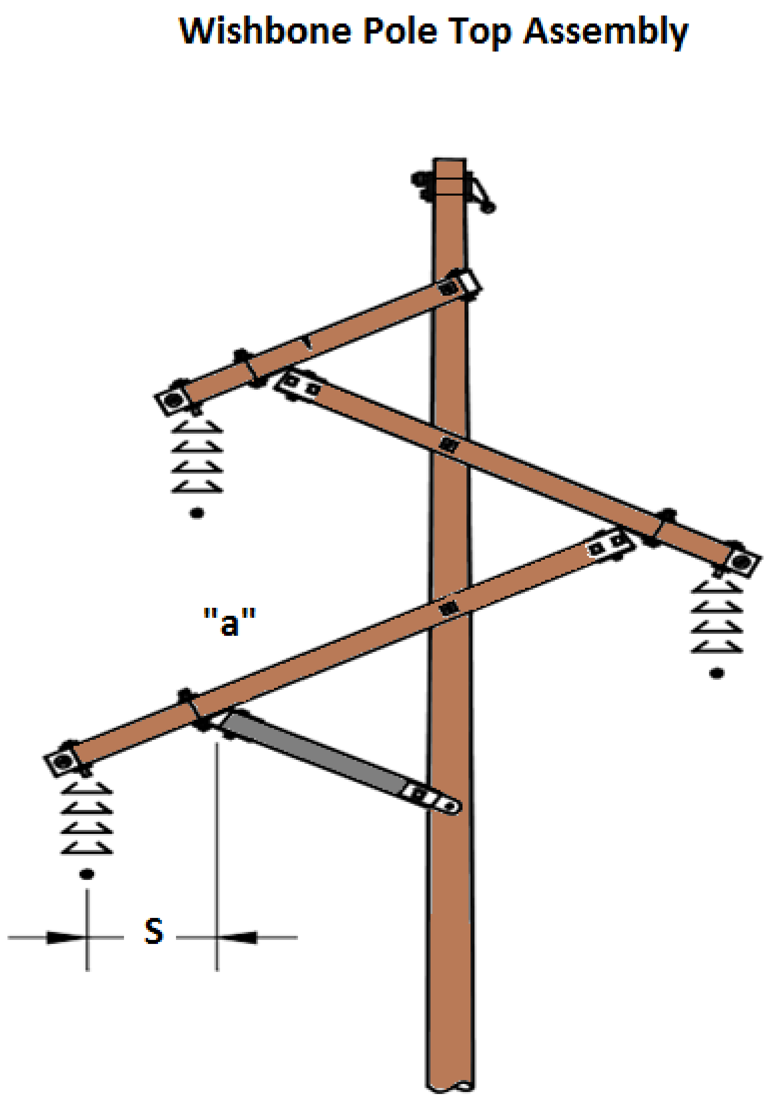

Figure 9

Since the crossarms of the wishbone are not horizontal, the vertical span is related to the horizontal span. The maximum vertical load the wishbone single crossarm assembly can withstand is 3,400 lbs. at any conductor position. By calculating moments at point "a" on the assembly, horizontal and vertical spans are related. Span limited by pole strength are calculated in the same manner as the post and suspension insulator structures.

Example of Span Calculations for Wishbone Pole Top Assemblies

Determine the maximum horizontal and vertical spans for the pole top assembly of the 69 kV wishbone pole top assembly (Figure 10).

Wishbone Structure Example

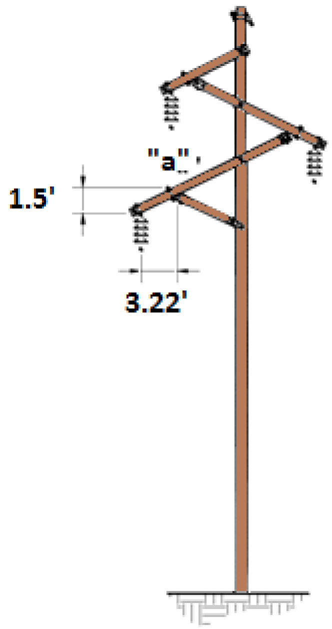

Figure 10

<u>Given</u>:
Loadings: NESC heavy loading district
Wires:
Conductor: 266.8 kcmil, 26/7 ACSR (Partridge)
OHGW: 3/8" HSS
Pole: S.Y.P. (70-1)

Moment capacity of crossarm at "a":

M_a = Maximum vertical load
M_a = 3,400 * 3.22 = 10,950 ft-lbs.

Horizontal and vertical span:

The relationship between the horizontal and vertical spans is obtained by summing moments about point 'a'.

$$(LF * p_c * 1.5' * HS) + (LF * w_c * 3.22' * VS) + (LF * w_i * 3.22') = \phi M_a$$

$$(2.5 * 0.5473 * 1.5 * HS) + (1.5 * 1.0776 * 3.22 * VS) + (1.5 * 50 * 3.22) = 0.50 * 10{,}950 \text{ ft-lbs.}$$

$$2.05HS + 5.21VS = 5234 \text{ ft-lbs.}$$

$$VS = -0.393HS + 1004.6$$

This equation defines the graph shown in Figure 11.

For HS = VS, Span = 720 ft.

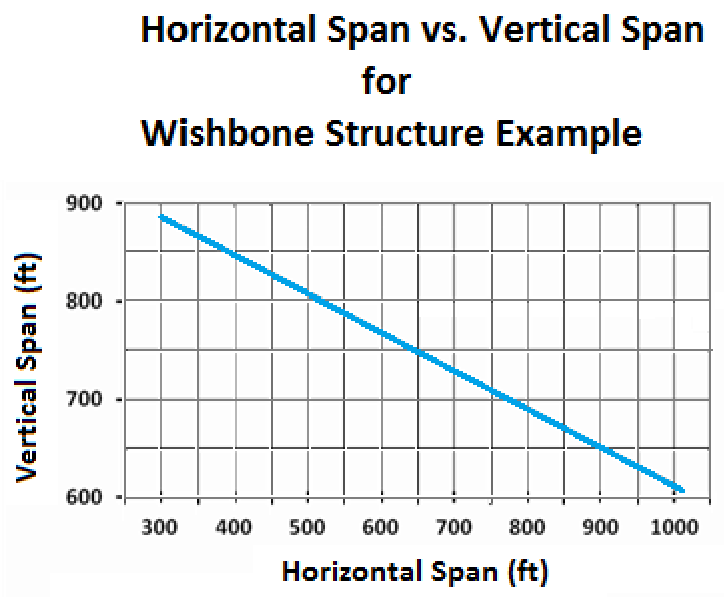

Figure 11

Span Calculations for Tangent Upswept Pole Top Assembly

These assemblies have steel upswept arms. With these arms, vertical spans are related to horizontal spans and a graph can be made to relate horizontal and vertical spans. Spans limited by pole strength are calculated in the same manner as the Tangent Post and Tangent Suspension structures.

Example of Span Calculations for Steel Davit Arm Construction

For the 138 kV structure in Figure 12, plot the horizontal versus vertical span for steel davit arms.

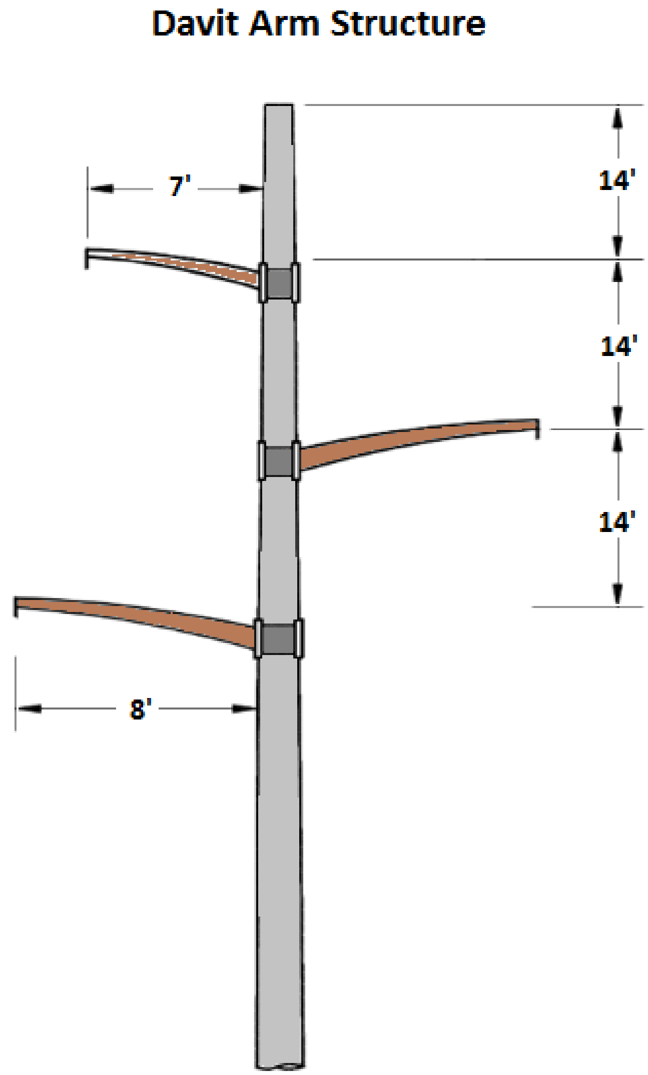

Figure 12

Given:

Loadings: NESC Heavy Loading
Extreme Wind 19 PSF on the wires

Wires:
Conductor: 477 kcmil, 26/7 ACSR
OHGW: 3/8” H.S.S
Pole: S.Y.P. (70-1)

Conductor loads:	Heavy Ldg	Extreme wind	EI&W
Transverse	0.6193	1.3585	0.8808
Vertical	1.5014	0.6570	2.4092

Manufacturers catalog data for crossarms:

Rated Ultimate Vert. load (Wc)	s, Length	R, Rise
3000	8.0	2.7
3000	7.0	2.5

Weight of insulators (Wi) = 102 lbs

For the 8.0' davit arm, the moment capacity of the arm at the pole (Figure 13):

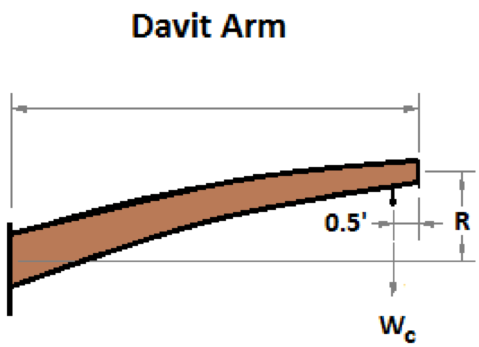

Figure 13

M_a = Maximum vertical load

$M_a = 3000 * (8.0 – 0.5)$

M_a = 22,500 ft-lbs.

An equation for the vertical and horizontal spans can be developed. Since the arm is steel, a strength factor (ϕ) of 1.0 is used.

$(LF * p_c * 2.7' * HS) + (LF * w_c * 7.5' * VS) + (LF * w_i * 7.5') = \phi * M_a$

(2.5 * 0.6193 * 2.7*HS) + (1.5 * 1.5014 * 7.5*VS) + (1.5 * 102 * 7.5) = 1.0 * 22,500 ft-lbs.

4.1803 HS + 16.89 VS = 21,352.5 ft-lbs.

VS = -0.25 HS + 1264.2

For the 7.0' davit arm, the moment capacity of the arm at pole:

$M_a = W_c * s$
$= 3000 * 6.5$
$= 19{,}500$ ft-lbs

An equation for the vertical and horizontal spans can be developed:

$(2.5*0.6193*2.5\ HS) + (1.5*1.5014*6.5*VS) + (1.5*102*6.5) = 1.0 * 19{,}500$ ft-lbs.

$3.87\ HS + 14.64\ VS = 18{,}505.5$ ft-lbs.

$VS = -0.26\ HS + 1264.0$

In this example for the NESC heavy loading district loads, the magnitude of the vertical span is not sensitive to the horizontal span (as shown in Figure 14). For horizontal spans between 300 and 1000 feet, the vertical span for the 8 foot arm as well as the 7 foot arm should be limited to 1004 feet (for design purposes, use 1000 feet). Spans limited by the extreme winds are not a factor in this example.

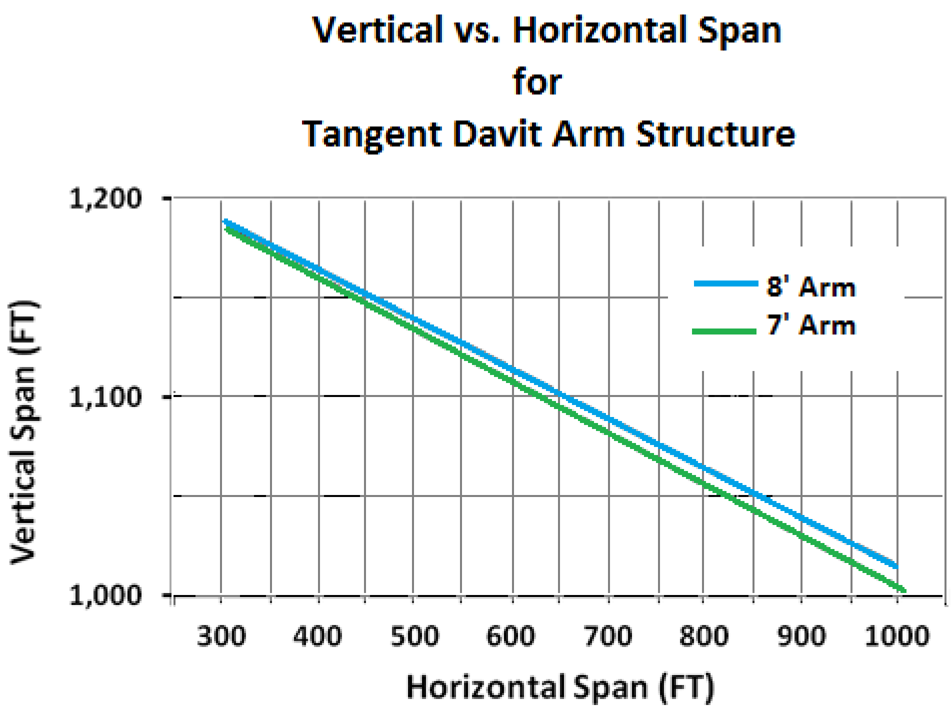

Figure 14

Chapter 2

Design Calculations for Wood H-Frame Structures

There are various techniques available for analysis of H-frame structures:

- Classical indeterminate structural analysis.
- Matrix methods of structural analysis.
- Approximate methods.
- Finite element analysis

Computer-aided design programs are available that accomplish finite element analyses for structural design. These programs are acceptable for use as long as they are in accordance with NESC design criteria. As with any design methodology, use of these programs must be performed by competent engineering personnel.

In analyzing a statically indeterminate structure by approximate procedures, one assumption is made for each degree of indeterminacy. These assumptions are based on logical interpretations of how the structure will react to a given loading. For the H-frame with knee and V-braces, we can assume that the structure will behave as shown in Figure 15.

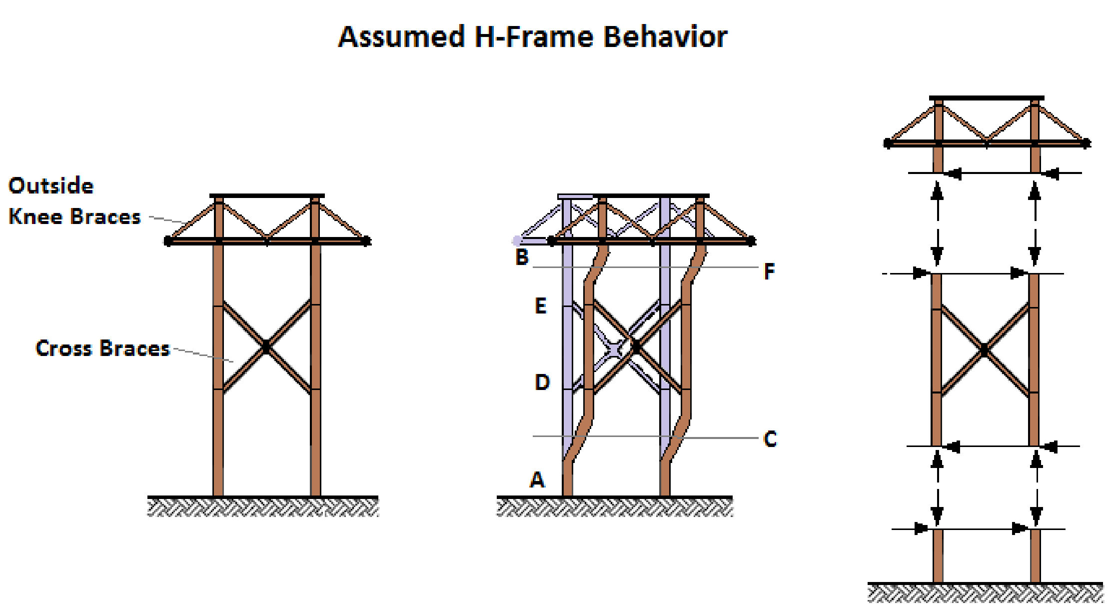

Figure 15

At some point in the poles, there will be an inflection point (a point of zero moment). If the pole or column is uniform in cross section, it is common to assume that the inflection point is located midway between points of bracing. (See Figure 15). However, since the pole is tapered, the following relationship may be used to determine the location of the inflection point. See Figure 16.

$$\frac{x_o}{x} = \frac{C_A * (2 * C_A + C_D)}{2 * (C_A^2 + C_A * C_D + C_D^2)}$$

Where,

C_A = circumference at base

C_D = circumference at top

Location of Point of Contraflexure

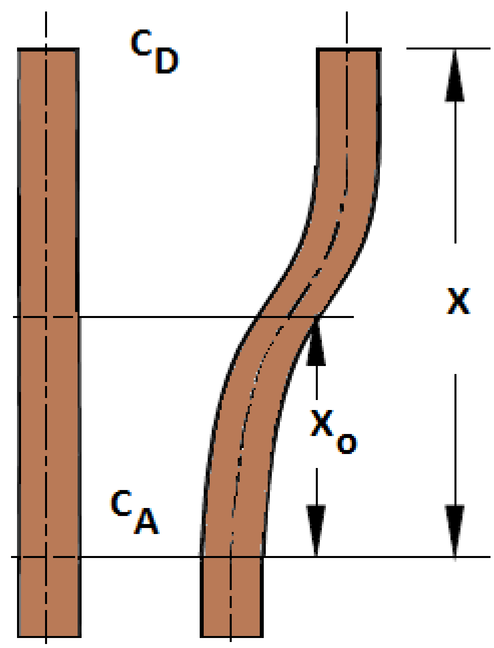

Figure 16

By applying the same reasoning, the inflection point can be located on the other column. Locating the inflection point on each column, and hence the point of zero moment, entails two assumptions for the frame. Since the frame is statically indeterminate to the third degree, a third assumption has to be made. A common third assumption is that the shear in the columns is distributed equally at the inflection points. The shear in the columns is equal to the horizontal force on the structure above the level under consideration.

For a less rigid support, the inflection point moves toward the less rigid support. Two conclusions can be made:

- For a pole rotating in the ground, the inflection point "C" below the crossbraces, is lowered. The lowering of the inflection point increasing the moment induced in the pole at the connection of the lower crossbrace. Since the amount of rotation of a base is difficult to determine, the usual design approach is to always assume a rigid base.

- For H-frames with outside kneebraces only, the point of inflection ‘F’ above the crossbrace (shown in Figure 15) is higher than the point of inflection for four kneebraces. This higher point of inflection increases the moment in the pole at the upper crossbrace pole connection. For the H-frame with outside kneebraces only, the designer may make one of two assumptions:
    - When determining induced moments in the poles, the outside kneebraces are ignored and no point of inflection exists between the crossbrace and the crossarm. This is a conservative assumption and assumes that the purpose of outside braces is to increase vertical spans only.
    - It can be assumed that the point of inflection occurs at the crossarm. This assumption will be used in the equations and examples which follow.

Crossbraces

The primary purpose of wood X-bracing for H-frame type structures is to increase horizontal spans by increasing structure strength. Additional benefits achieved by crossbracing include possible reduction of right-of-way costs by eliminating some guys and reduction of lateral earth pressures. For an efficient design, several calculations should be made in order to correctly locate the crossbrace.

The theoretical maximum tensile or compressive load which the wood crossbrace will be able to sustain will largely be dependent on the capacity of the wood brace to sustain a compressive load. Common crossbrace dimensions are 3-3/8" x 4-3/8" for the 115 kV structure and 3-3/8" x 5-3/8" for 138 kV and 161 kV structures. The dimensions of this X-brace are 3-5/8" x 7-1/2" (minimum). See Figure 17.

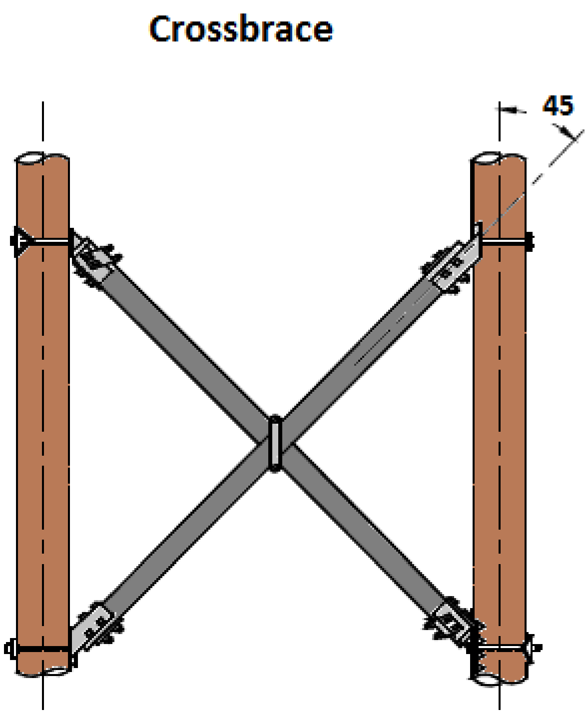

Figure 17

The maximum compressive load which a wood X-brace is able to sustain is determined by:

$$\mathbf{P_{cr}} = \frac{\mathbf{A} * \boldsymbol{\pi}^2 * \mathbf{E}}{(\frac{\mathbf{kl}}{\mathbf{r}})^2}$$

Where,
P_{cr} = maximum compressive load, lbs.
A = area, in^2
E = modulus of elasticity, psi.
$k\ell$ = effective unbraced length, in.

r = radius of gyration, in. which will give the maximum $k\ell/r$ ratio; $k\ell$ and r must be compatible for the same axis.

For an assumed one foot diameter pole, the following theoretical values apply:

Table 3 Crossbrace Capacities						
Crossbrace	A Area (in^2)	r Least Radius of Gyration (in.)	L Distance CL to CL of Poles (ft.)	0.5L / 0.707 less 1' for Pole Dia. (in.)	kl / r	P_{cr} for $E = 1.8 * 10^6$ (lbs.)
3-3/8" x 4-3/8"	14.77	0.9743	12.5	97.6	100.2	26,100
3-3/8" x 5-3/8"	18.14	0.9743	15.5	123.1	126.3	20,200
3-5/8" x 7-1/2"	27.19	1.05	19.5	157	149.5	21,600

The calculations included in Table 3 do not reflect the capacity of the hardware. Double Armed and Braced Type Crossarm Assemblies, and Double Armed and Braced Type Crossarm Assemblies require X-braces to withstand a tension or compression loading of 20,000 lbs. This ultimate value correlates with the above theoretical ultimate loads in the table. It is recommended that 20,000 lbs. (ultimate) be used for design purposes, since this value assures one that the crossbrace will sustain the indicated load.

For a 115 kV structure it is recommended that 20,000 lbs. be used as the ultimate load the crossbrace is able to sustain. The hardware for the crossbrace is the same as the hardware used with 138 kV and 161 kV structures.

V-Braces

The primary purpose of two V-braces on the outside of the poles is to increase vertical spans. Two V-braces on the inside will increase horizontal spans. Four V-braces increase both horizontal and vertical spans. The various bracing arrangements and their designations for 161 kV structures are shown in Figure 18.

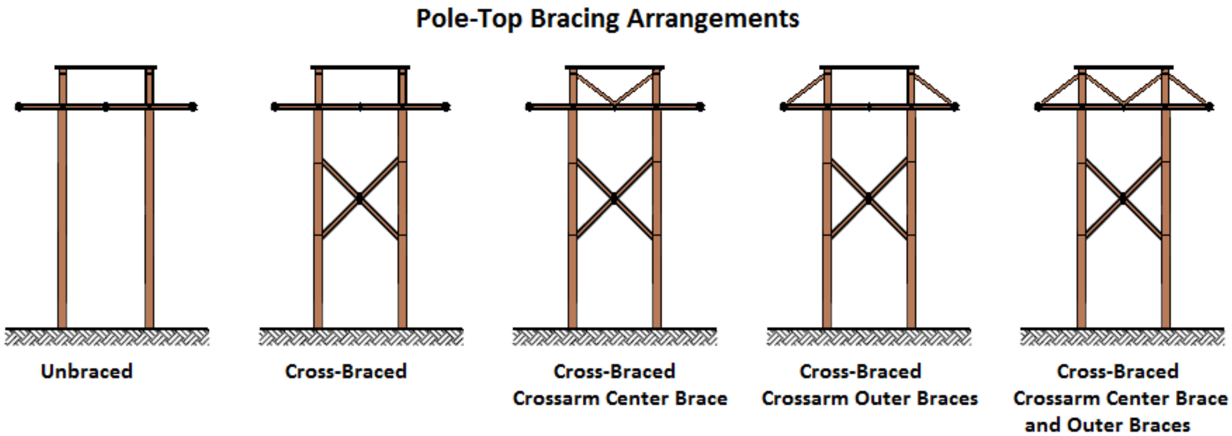

Figure 18

Double Armed and Braced Type Crossarm Assemblies (138 kV and 161 kV) specifies the following minimum strength requirements for the various pole top assemblies:

- Maximum vertical load (at any conductor position)
  - Unbraced 8,000 lbs.
  - Two outside X-arm Braces 14,000 lbs.
  - Four X-arm Braces 14,000 lbs.

- Maximum transverse conductor load (total)
  - Two outside X-arm Braces 15,000 lbs.
  - Four X-arm Braces 15,000 lbs.

- Maximum tension or compression in V-brace" 20,000 lbs.

Double Armed and Braced Type Crossarm Assemblies (230 kV) specifies the following minimum strength requirements for the 230 kV H-frame pole top assembly:

- Maximum vertical load (at any conductor position): 10,000 lbs.

- Maximum transverse conductor load (total): 15,000 lbs.

- Maximum tension or compression in V-brace: 20,000 lbs.

When determining maximum vertical and horizontal spans as limited by H-frame top assemblies, the above minimum strengths may be used as guidance.

Structure Analysis of H-frames

Equations will be shown shortly for calculating forces in the various members of H-frame structures. As part of the structural analysis, span limitations due to strength of the pole top assembly should be considered and suggested methods follow. Appropriate load factors and strength factors should be applied in the respective equations.

Outside V-Braces

An H-frame structure with two outside V-braces in Figure 19 needs further explanation. A structure with two outside V-braces has less rigidity above the crossbrace than a structure with four V-braces. The location of the point of contraflexure is difficult to determine. The calculation for the moment (M_E) at the top of the crossbrace assumes that the point of contraflexure exists at the crossarm. However, to determine span limitations due to strength of the pole top assembly, a point of contraflexure is assumed between the top of the crossbrace and the crossarm. The maximum vertical span is determined for the maximum horizontal span.

> Note: The point of contraflexure is assumed to be between the top of the crossbrace and the crossarm when determining span limitations due to strength of the pole top assembly.

Pole Top Assembly with Two Outside Braces

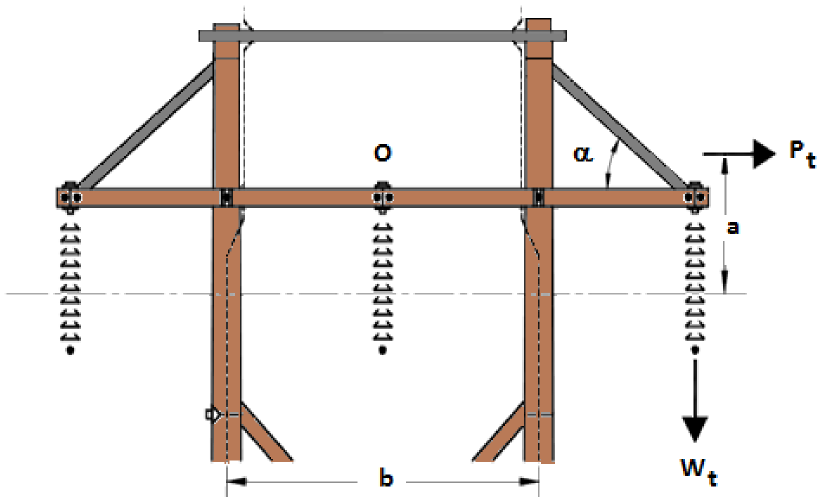

Figure 19

Ultimate force in the brace is:

$$\frac{\mathbf{LF * W_t}}{\mathbf{sin(\alpha)}} + \frac{\mathbf{LF * P_t * a}}{\mathbf{b * sin(\alpha)}} \leq \boldsymbol{\phi} * \mathbf{20,000\ lbs}$$

Where,

W_t = total vertical load at the phase wire, locations, lbs,

$W_t = VS * w_c + W_i$,

VS = vertical span, ft.

w_c = weight load per foot of conductor, lbs./ft.

W_i = total weight of the insulators, lbs.

P_t = total transverse load, lbs.

$P_t = HS * (3p_c+2p_g)$

where,

HS = horizontal span, ft.

p_c = wind load per foot of conductor, lbs./ft.

p_g=wind load per foot of overhead ground wire, lbs./ft.

a = distance from the point of contraflexure to equivalent force, ft.

b = distance between poles, ft.

LF = load factor

α = angle the brace makes with the crossarm

Two Inside V-Braces

Pole bending moment, uplift, and force in the X-brace may be calculated in the same manner as when four braces are used. Crossarm strength controls the maximum vertical span.

Force in the braces is:

$$\frac{\mathbf{LF * W_t}}{\mathbf{2 * sin(\alpha)}} + \frac{\mathbf{LF * P_t * a}}{\mathbf{b * sin(\alpha)}} \leq \boldsymbol{\phi} * \mathbf{20,000\ lbs}$$

Crossarm bending moment, $(\phi)*M_o$ is:

$$\boldsymbol{\phi} * \mathbf{M_o} = \frac{\mathbf{LF * W_t * b}}{\mathbf{2}}$$

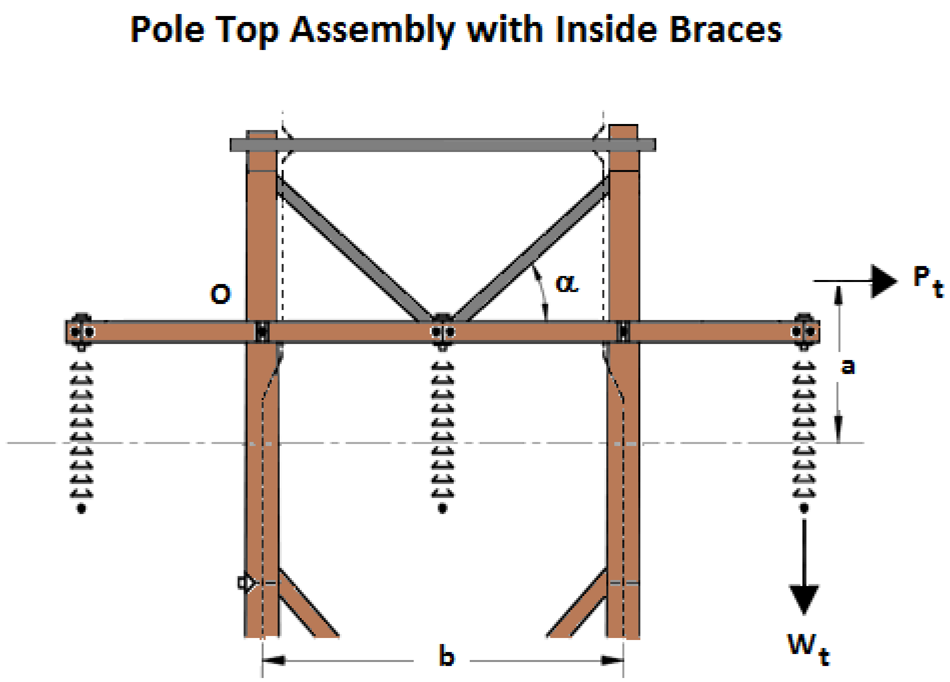

Figure 20

Four V-Braces

The following equations can be used to determine the maximum vertical span as limited by four V-braces, given the maximum horizontal span:

For four V-braces, force in the outside braces is:

$$\frac{LF * W_t}{\sin(\alpha)} \le (\phi) * 20{,}000\text{lbs}$$

Force in the inside braces is:

$$\frac{LF * W_t}{2 * \sin(\alpha)} + \frac{LF * P_t * a}{b * \sin(\alpha)} \le \phi * 20{,}000 \text{ lbs}$$

Structural Analysis Equations

What follows are six different H-frame structure configurations and the calculations associated with each structure type.

Equations for Structure 1 (Figure 21)

For this structure, the horizontal span is reduced by 10% to take into account P-delta (P-δ) moments

$$\mathbf{HS_A = (\phi * M_A) - \frac{LF * F * h^2 * (2 * d_t + d_a)}{6} / (\frac{LF * p_t * h_1}{2} * 0.90)}$$

$$\mathbf{R_A = LF * (W_g + \frac{3}{2 * W_t} + W_p)}$$

$$\mathbf{VS = \frac{\phi * M_a - (LF * W_i * S)}{w_c * S * LF}}$$

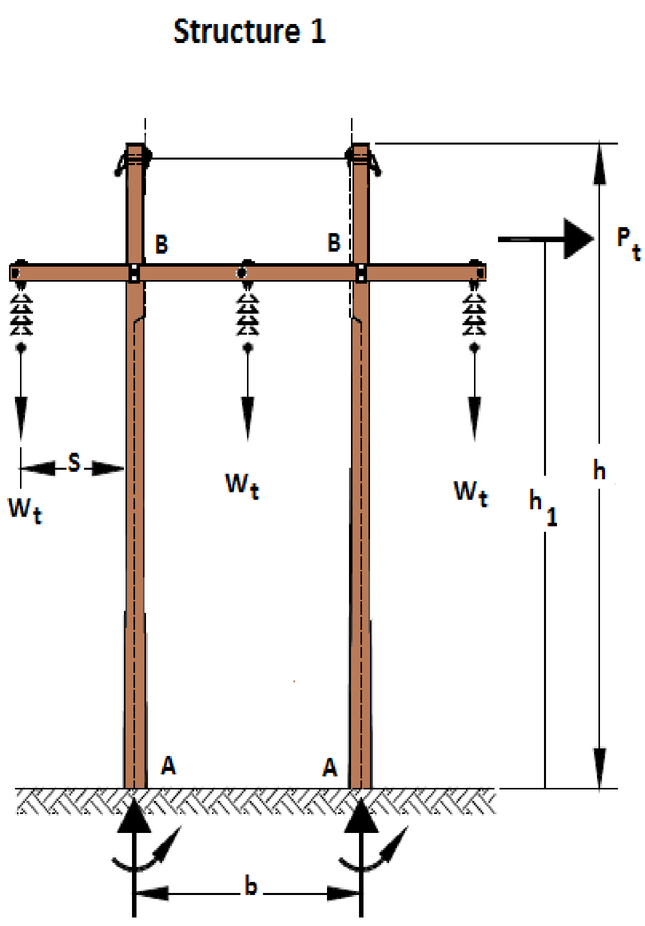

Figure 21

Equations for Structure 2 (Figure 22).

$$\mathbf{HS_B = (\phi * M_B) - \frac{LF * F * y_1^2 * (2 * d_t + d_b)}{6} / (\frac{LF * p_t * y_1}{2})}$$

$$\mathbf{HS_E = (\phi * M_E) - \frac{LF * F * y^2 * (2 * d_t + d_e)}{6} / (\frac{LF * p_t * y_0}{2})}$$

$$\mathbf{HS_D = (\phi * M_D) - \frac{LF * F * (h - x_0) * x_1 * (2 * d_t + d_c)}{6} / (\frac{LF * p_t * x_1}{2})}$$

$$\mathbf{HS_A = (\phi * M_A) - \frac{LF * F * (h - x_0) * x_0 * (2 * d_t + d_c)}{6} / (\frac{LF * p_t * x_0}{2})}$$

For crossbrace:

$$\mathbf{HS_x = \frac{\phi * 28,300 * b - 2 * LF * F * (h - x_0)^2 * (2 * d_t + d_c)}{6} / (LF * p_t * h_2)}$$

For uplift:

$$\mathbf{HS * p_t * h_2 - (VS * w_g * b) - (1.5 * VS * w_c * b) = (W_1 * b) + (W_p * b) + X - Y}$$

For bearing:

$$\mathbf{HS * p_t * h_2 + (VS * w_g * b) + (1.5 * VS * w_c * b) = (W_2 * b) - (W_p * b) + X - Y + (W_1 * b)}$$

Structure 2

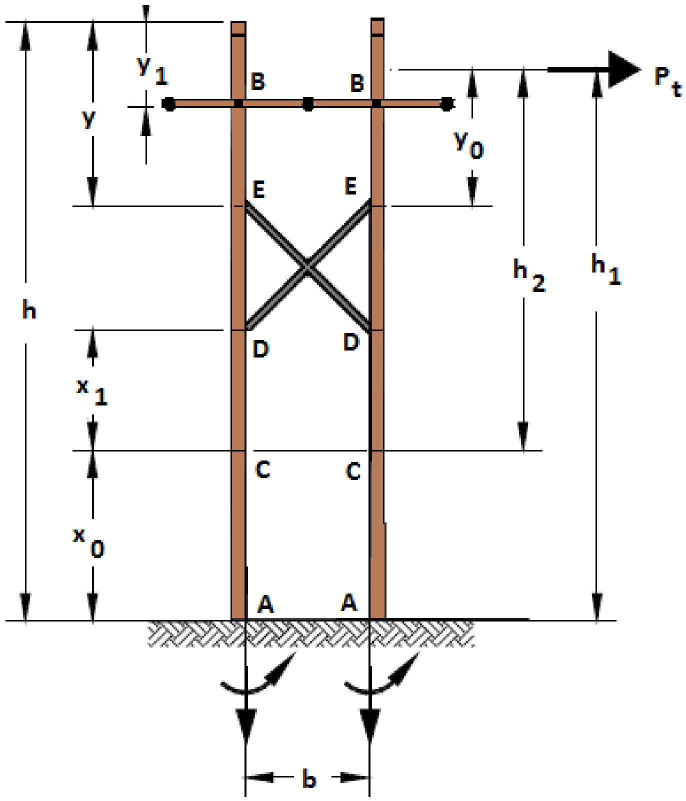

Figure 22

Where,

$$W_1 = \frac{F_S * D_e * d_{avg} * \pi}{SF}$$

$$W_2 = \frac{\frac{\pi * d_{bt}^2}{4} * Q_u}{SF}$$

$$X = F \;\; * (h - x_o) * (d_t + d_c) * (x_o)$$

$$Y = \frac{2 * f * h^2 * (2 * d_t + d_a)}{6}$$

Equations for Structure 3 (Figure 23)

$$\mathbf{HS_E} = (\boldsymbol{\phi} * \mathbf{M_E}) - \frac{\mathbf{LF * F * y_1 * z * (d_t + d_b)}}{\mathbf{2}} / (\frac{\mathbf{LF * p_t * z}}{\mathbf{2}})$$

HS_D and HS_A are the same as Structure #2

For crossbrace, uplift and bearing: same as structure #2

Structure 3

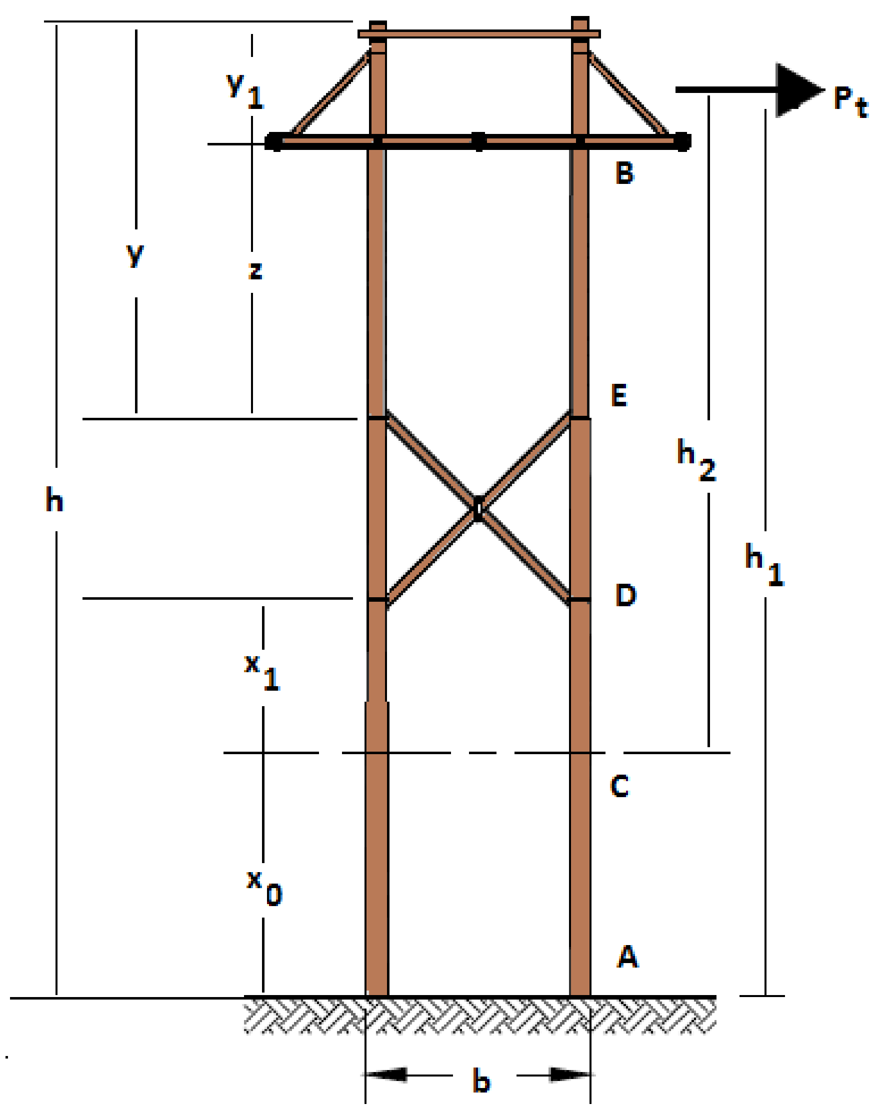

Figure 23

Equations for Structure 4 (Figure 24.

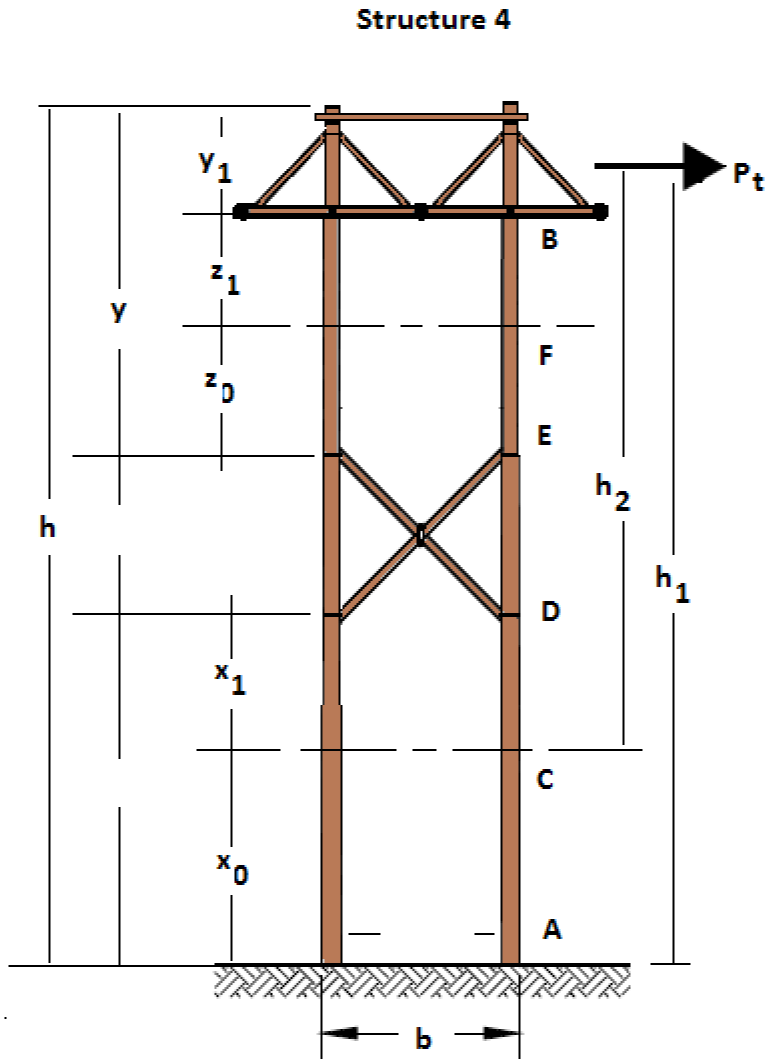

Figure 24

$$HS_B = (\phi * M_B) - \frac{LF * F * (y - z_0) * (d_t + d_f)}{2} / (\frac{LF * p_t * z_1}{2})$$

$$HS_E = (\phi * M_E) - \frac{LF * F * (y - z_0) * (d_t + d_f)}{2} / (\frac{LF * p_t * z_0}{2})$$

HS_D, HS_A are the same as structure #2.

For uplift and bearing: same as structure #2.

$$\mathbf{HS_x = \frac{\phi * 28,300 * b - U + V}{LF * p_t * (h_2 - a)}}$$

Where,

$$\mathbf{U = \frac{2 * LF * F * (h - x_0)^2 * (2 * d_t + d_c)}{6}}$$

$$\mathbf{V = \frac{2 * LF * F * (y - z_0)^2 * (2 * d_t + d_f)}{6}}$$

28,300 lb is the maximum load of the crossbrace equivalent to (20,000lb) * 2 * cos(45°)

Equations for Structure 5 (Figure 25).

For crossbrace:

$$HS_x = \frac{\phi * 56,500 * b - 2 * LF * F * (h - x_0)^2 * (2 * d_t + d_c)}{6} / (LF * p_t * h_2)$$

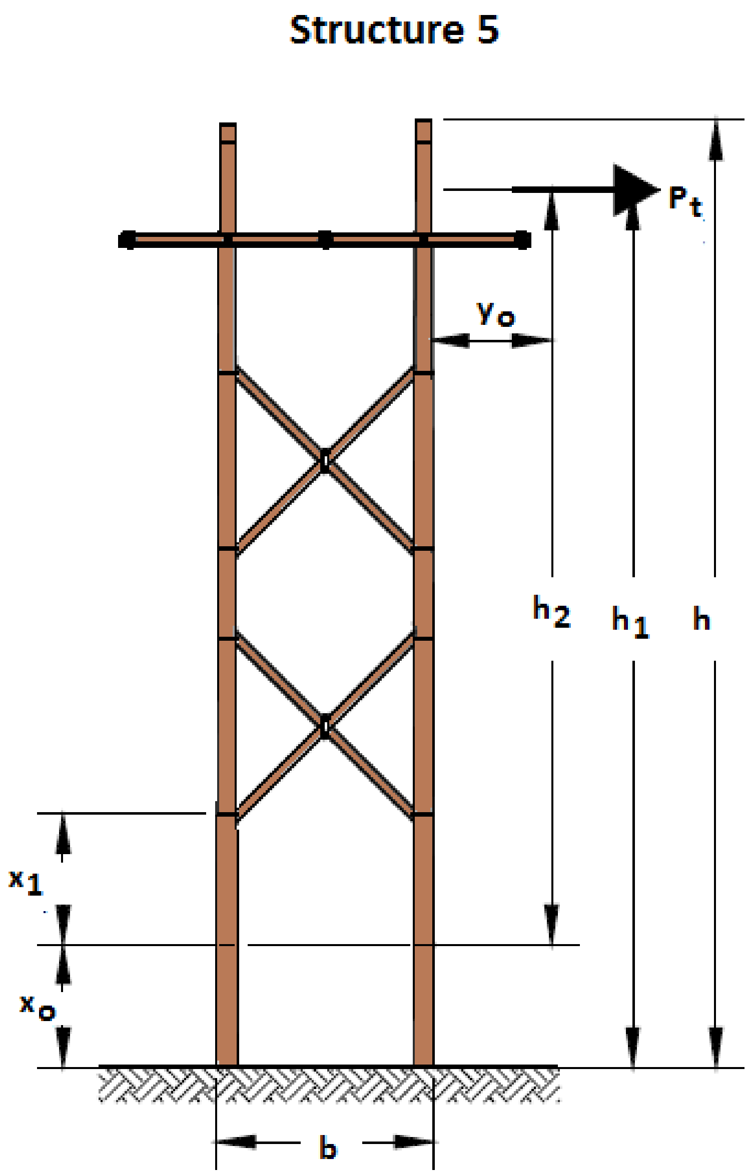

Figure 25

Equations for Structure 6 (Figure 26).

For crossbrace:

$$\mathbf{HS_x} = \frac{\boldsymbol{\phi} * \mathbf{56,500} * \mathbf{b} - \mathbf{U} + \mathbf{V}}{\mathbf{LF} * \mathbf{p_t} * (\mathbf{h_2} - \mathbf{a})}$$

Where,
U, V are the same as structure #4

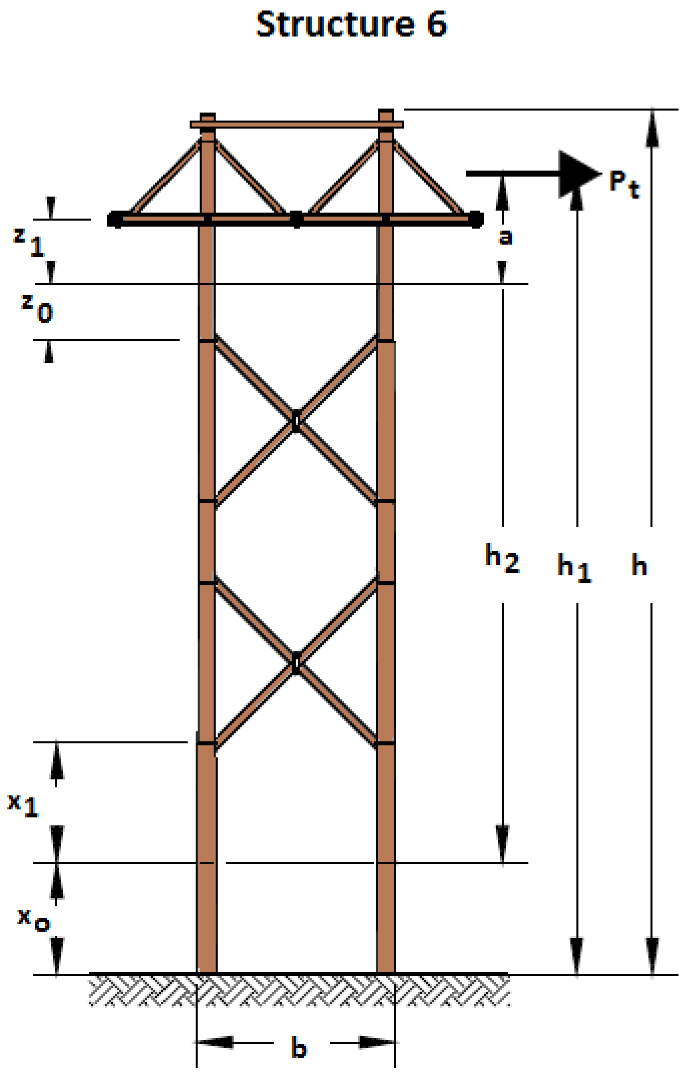

Figure 26

Example of an H-frame Analysis

For the 161 kV structure shown in Figure 27, determine the horizontal span based on structure strength and uplift and plot the horizontal versus vertical span for the pole top assembly.

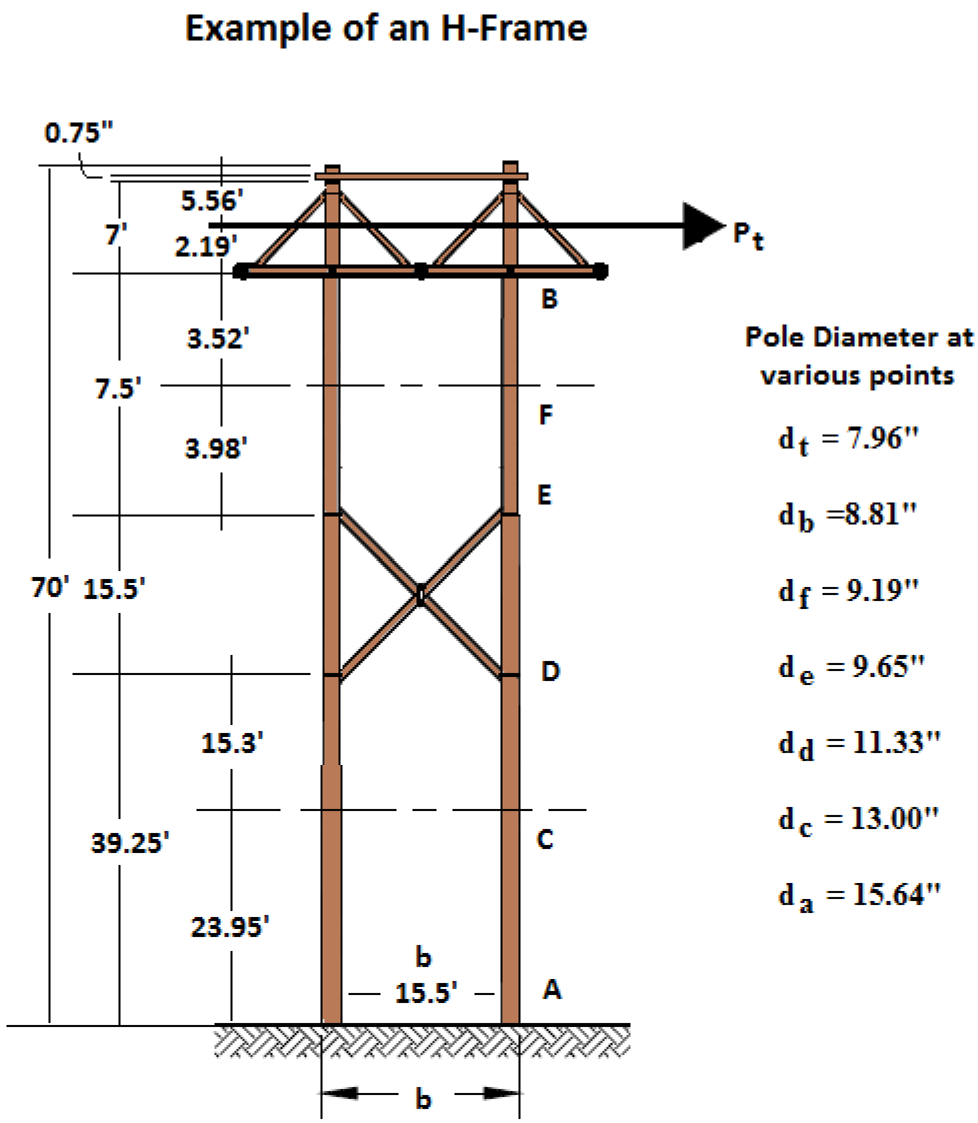

Figure 27

Note: See Figures 24 and 27 for this example problem.

<u>Given</u>:
NESC heavy loading
Extreme winds: 19 PSF on the wires and
22 PSF on the structure
Extreme ice: 1" radial ice
Extreme ice with concurrent wind: 1” radial ice with 4 PSF wind

Pole: Douglas fir 80-2
Conductor: ACSR 795 kcmil 26/7
OHGW: 7/16 E.H.S.
Ruling Span: 800 ft.

Conductor Loads	Heavy Ldg District	Extreme Wind	Heavy Ice	EI&W
Transverse Loads	0.7027 lbs./ft.	1.7543 lbs./ft.	0	1.0360
Vertical Loads	2.0938 lbs./ft.	1.0940 lbs./ft.	3.7154 lbs./ft.	3.7154
Tension	10,400 lbs.	---	14,000 lbs	XXX

OHGW Loads	Heavy Ldg District	Extreme Wind	Heavy Ice	EI&W
Transverse Loads	0.4783 lbs./ft.	0.6888 lbs./ft.	0	0.8116
Vertical Loads	0.9803 lbs./ft.	0.3990 lbs./ft.	2.1835 lbs./ft.	2.1835
Tension	5,900 lbs.	---	7,500 lbs.	XXX

The soil is average. Presumptive skin friction, Fs, of 250 PSF for predominantly dry soil areas and using native backfill; 500 PSF when aggregate backfill is used.

Maximum horizontal span based on structure strength:

Step 1.
Equivalent Load pt:

$$p_t = 2p_g + 3p_c$$

$$p_t = 2 * 0.4783 + 3 * 0.7027$$

$$p_t = 3.065 \text{ lbs / ft.}$$

Step 2.
Determine location of equivalent load, p_t:

$$\text{Distance from Top} = \frac{2 * p_g * 0.75 + 3 * p_c * 7.75}{P_t}$$

$$\text{Distance from Top} = 5.56 \text{ ft.}$$

Step 3.
Determine location of x_0, x_1, and z_1 for the X-brace location shown. The ratios for x_0/x_1 or z_0/z determined by the following equation,

$$\mathbf{x_0/x_1 = 0.207 * (\frac{d_d}{d_a})^2 - 0.7382 * (\frac{d_d}{d_a}) + 1.0326}$$

For x_0, x_1:

d_d/d_a = 11.33 / 15.64 = 0.72

Therefore,

$x_0/x_1 = 0.207(0.72)^2 - 0.7382(0.72) + 1.0326$

$x_0/x = 0.61$

The bottom of the brace (Point “D” in Figure 27) is 39.25 feet, therefore,

$x_0 = 0.61 * 39.25$

x_0 = 23.9 ft.

And x_1 is (39.25 – 23.9) = 15.3 ft.

At this location the pole diamater is,

d_c = 13.0 in.

For z_o, z_1 :

d_d / d_e = 8.81 / 9.65 =0.91

Therefore,

$z_0/z = 0.207(0.91)^2 - 0.7382(0.91) + 1.0326$

$z_0 / z = 0.53$

The distance from the brace to the crossarm is 7.5 feet so,

$z_o = 0.53 * 7.5$

z_o = 3.98 ft.

and, $z_1 = (7.5 - 3.98) = 3.52$ ft.

At this location the pole diameter is,

$d_f = 9.19$ in.

Step 4.

Find the horizontal span limited by pole strength at B (see Figure 26) using,

$$\mathbf{HS_B = (\phi * M_B) - \frac{LF * F * (y - z_0) * (d_t + d_f) * z_1}{2} / (\frac{LF * p_t * z_1}{2})}$$

$M_B = 44{,}700$ ft −lbs.

$y = 7.5' + 7.0' + 0.50' = 15.25$ ft.

$$HS_B = (0.65 * 44{,}700) - \frac{2.5 * 4 * (15.25 - 3.98) * (0.663 + 0.766) * 3.52}{2} / (\frac{2.5 * 3.065 * 3.52}{2})$$

$HS_B = 2{,}133$ ft.

Step 5.

Horizontal span limited by pole strength at E:

$$\mathbf{HS_E = (\phi * M_E) - \frac{LF * F * (y - z_0) * (d_t + d_f) * z_0}{2} / (\frac{LF * p_t * z_0}{2})}$$

$M_E = M_{cap} – M_{bh}$

$M_{cap} = 58{,}800$ ft-lbs

Note: M_{bh} is found from manufacturer's data. In this example use 8,400 ft-lbs.

$M_E = 58{,}800 - 8{,}400$ ft-lbs.

$M_E = 50{,}400$ ft-lbs

$$HS_E = (0.65 * 50{,}400) - \frac{2.5 * 4 * (15.25 - 3.98) * (0.663 + 0.766) * 3.98}{2} / (\frac{2.5 * 3.065 * 3.98}{2})$$

HS_E = 2,127 ft.

Step 6
For horizontal span limited by pole strength at locations D and A, similar calculations can be made. The results are as follows:

HS_D = 811 ft
HS_A =1664 ft

Step 7.
For horizontal span limited by strength of the crossbrace:

$$\mathbf{HS_x = \frac{\phi * 28,300 * b - U + V}{OLF * p_t * (h_2 - a)}}$$

Where,

$$\mathbf{U = \frac{2 * LF * F * (h - x_0)^2 * (2 * d_t + d_c)}{6}}$$

$$U = \frac{2*2.5*4*(70-23.9)^2*(2*0.663+0.766)}{6}$$

U = 17,065 ft-lbs

$$\mathbf{V = \frac{2 * LF * F * (y - z_0)^2 * (2 * d_t + d_f)}{6}}$$

$$V = \frac{2*2.5*4*(15.25-3.98)^2*(2*0.663+0.766)}{6}$$

V= 885 ft-lbs

$$HS_x = \frac{0.65 * 28{,}300 * 15.5 - 17{,}065 + 885}{2.5 * 3.065 * 34.78)}$$

HS_X = 1009 ft.

Maximum span limited by pole top assembly:

Using,

$$\mathbf{VS} = \frac{(\boldsymbol{\phi}) * \mathbf{20,000lbs} * \mathbf{sin}(\boldsymbol{\alpha}) - (\mathbf{LF} * \mathbf{W_i})}{\mathbf{LF} * \mathbf{W_t}} \leq$$

$$\text{VS} = \frac{0.65 * 20{,}000 * \sin(39) - (1.5 * 135)}{2.0938 * 1.5}$$

VS = 2,543 ft.

Using,

$$\frac{\mathbf{LF} * \mathbf{W_c} * \boldsymbol{VS}}{\mathbf{2} * \mathbf{sin}(\boldsymbol{\alpha})} + \frac{\mathbf{LF} * \mathbf{P_t} * \mathbf{a} * \mathbf{HS}}{\mathbf{b} * \mathbf{sin}(\boldsymbol{\alpha})} \leq \boldsymbol{\phi} * \mathbf{20,000\ lbs}$$

Point “a” is found from,

- Distance from top of pole to cross-arm = 7.75’
- Distance from top of pole to Pt = 5.56’
- Add z_1 (3.52’)
- Point “a” = 7.75’ - 5.56’ + 3.52’ = 5.71’

$$\frac{1.5 * 2.0938 * \text{VS}}{2 * \sin(39)} + \frac{2.5 * 3.065 * (5.71) * \text{HS}}{15.5 * \sin(39)} \leq 0.65 * 20{,}000\ \text{lbs}$$

2.49VS + 4.48HS ≤ 13,000 lbs.

(For VS equal to the HS, the vertical span is 1,865 ft.)

<u>Maximum span limited by uplift</u>:

Assume dry native backfill, safety factor of 4.

(HS * p_t * h_2) – (VS * w_g *b) – (1.5VS * w_c * b) = (W_1 * b) + (W_p * b) + X −Y

Where,
Pt = 3.065
h_2 = 70’ – 5.56’ - 23.9’ = 40.54’
w_g = 2.1835
b = 15.5’

w_c = 3.715
D_e = 10’

d_{avg} = diameter from height at guy attachment (d_a) to diameter at butt of pole (d_{bt})
d_a = 15.64”
d_{bt} = 16.48”
d_{avg} = 11.80” or 0.98 ft.

$$\mathbf{W_1 = \frac{F_s * D_e * d_{avg} * \pi}{SF}}$$

$$W_1 = \frac{250 * 10 * 0.98' * 3.14}{4}$$

W_1 = 1,923 lbs.

W_p = Wt. of one pole and half the weight of pole top assembly and crossbrace.

W_p = 4200 +800/2 = 4,600 lbs.

$$\mathbf{X = F * (h - x_o) * (d_t + d_c) * x_o}$$

X = 4 * (70 – 23.9) * (7.76”+ 13.0”)/12 * 23.9’

X = 7705 ft-lbs.

$$\mathbf{Y = 2 * F * h^2 * (2d_t + d_a)/6 = 17{,}182\ ft - lbs.}$$

$Y = 2 * 4 * 70^2 * (2*7.96" + 15.64")/12/6$

Y= 17,182 ft −lbs.

The equation is as follows:

$$(HS * 3.065 * 40.54) - (VS * 0.98 * 15.5) - (1.5VS * 2.0938 * 15.5) = (1{,}923 * 15.5) + (4600 * 15.5) + 7{,}705 - 17{,}182$$

124.26HS − 63.9VS = 91,630

(For VS = 0, maximum HS = 737 ft.)

Check for extreme ice and concurrent wind. Span limitations based on pole strength and crossbrace strength is controlled by NESC Heavy Loading conditions. The unit conductor loads when load factors and strength factors accounted for, are greater for the Heavy Loading District load than for the EI&W as shown below.

	NESC > EI&W
Transverse load:	0.7027 * 2.5 / 0.65 > 1.0360 / 0.65 lbs/ft
	2.7027 > 1.5938 lbs/ft.

	NESC > EI&W
Vertical load:	2.0938 * 1.5 / 0.65 > 3.7154 lbs/ft
	4.8318 > 3.7154 lbs/ft

Check for Extreme Wind Conditions. Although span limitations based on pole strength and crossbrace strength is controlled by NESC Heavy Loading conditions, span limitations based on uplift is controlled by the extreme wind condition.

- For Dry Native Backfill**:** For an assumed safety factor of 1.5, the following equation result:

  191.8HS – 120.2VS = 141,307
  (For VS=0, maximum HS = 737 ft.)

- For Aggregate Backfill**:** For an assumed safety factor of 1.5, the following equation results:

  191.8HS – 120.2VS = 220,812
  (For VS=0, maximum HS = 1,151 ft.)

When considering uplift, it may be prudent to base calculations on the minimum vertical span as limited by insulator swing.

Summary of Span Limitations

- Horizontal Span limits**:**
  HS_A = 1664 ft.
  HS_D = 811 ft.

HS_E = 2127 ft.
HS_B = 2133 ft.
HS_x = 1009 ft.

- Dry native backfill:

For a VS = 0, the HS (limited by uplift) = 737 ft.

- Aggregate backfill:

For a VS = 0, the HS (limited by uplift) = 1,151 ft.

- Vertical Span limited by Heavy District Loads**:**

$VS_{poletop}$ = 1,865 ft., max. (For VS =HS)

A more efficient design could be achieved by moving the crossbrace.

Abbreviations

Listed below are key abbreviations used in this course.

A = area, in^2
a = distance from the point of contraflexure to equivalent force, ft.

b = distance between poles, ft.

C_A = circumference at base
C_D = circumference at top

d_1 = diameter of pole, inches, at height "h_1" above ground line
d_{pt} = diameter at h_1 above ground line
= diameter of pole at the resultant load, p_t, inches
d_a = diameter of pole at groundline, inches
d_{avg} = average diameter of pole between groundline and butt, ft.
d_{bt} = diameter of pole at butt, ft.
D_e = embedment depth
d_g = diameter of pole at groundline, inches
d_{ms} = diameter of pole at point of maximum stress (P_{oms}), inches
d_n = diameter at location "n," ft.
d_t = diameter of pole at top, inches

E = modulus of elasticity, psi.

F = wind pressure on a cylindrical surface, PSF
F_b = fiber stress, psi
f_s = calculated skin friction value, PSF

h = height of pole above groundline, feet
h_1 = Moment arm of P_t above groundline, feet
h_{1max} = distance from location of P_t to P_{oms}, feet
h_2 = distance from P_{oms} location to top of pole, feet
h_n = length as indicated, ft.
HS = horizontal span, feet

$k\ell$ = effective unbraced length, in.

LF = load factor

$M_A = F_b * S$, the ultimate moment capacity of the pole, ft-lbs.
M_g = the induced moment
M_n = moment capacity at the indicated location 'n', ft-lb.
$M_{p\text{-}\delta}$ = groundline moment due to pole deflection
M_{vo} = groundline moment due to unbalanced vertical load
M_{wc} = groundline moment due to wind on the wires
M_{wp} = groundline moment due to wind on the pole

p_c = wind load per foot of conductor, lbs./ft.
P_{cr} = critical buckling load
p_g=wind load per foot of overhead ground wire, lbs./ft.
P_t = total horizontal force per unit length due to wind on the conductors and overhead ground wire, lbs./ft.

Q_u = ultimate bearing resistance of the soil, PSF

r = radius of gyration, in. which will give the maximum $k\ell/r$ ratio;
R_n = reaction at the indicated location,"n," lbs.

S = section modulus of the pole
s_a , s_b ,and s_c = horizontal distances from center of pole to conductors
SF = Presumptive skin friction
s_g = Horizontal distance from center of pole to ground wire
s_n = distance as shown, ft.

V_n = induced axial force at the indicated location, lbs.
VS = vertical span, ft.

W_1 = total resistance due to skin friction around the embedded portion of the pole, lbs.
W_2 = total bearing resistance of the soil, lbs.
w_c = Weight of the conductor per unit length, lbs./ft
W_c = weight of conductors (plus ice, if any), lbs.
W_g = weight of OHGW (plus ice, if any), lbs.
w_g = weight of overhead groundwire per unit length, lbs./ft.
W_i = Insulator weight, lbs.
$W_{l\text{-}p}$ = weight of a line person
W_p = weight of pole, lbs.
W_r = total unit load
W_t = total vertical load at the phase wire, locations, lbs,

w_t = total weight per unit length of all wires, lbs./ft.

α = angle the brace makes with the crossarm

δ_{imp} = improved estimate of deflection of the structure, ft.
δ_{mag} = deflection magnifier, no units, (assume 1.15 initially)

ϕ = strength factor

U = dummy variable

V = dummy variable

X = dummy variable

Y = dummy variable

Summary

The primary purpose of this series of courses is to furnish engineering information for use in designing transmission lines. Good line design should result in high continuity of service, long life of physical equipment, low maintenance costs, and safe operation. These courses presents a generalized "how to" guide for the design of a high voltage transmission line.

The final volume in the series is concerned with the structural aspects of transmission line design and includes both single-pole structures and H-frame structures.

We hope you have enjoyed and benefited from this series of courses.

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